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Engineer Research and Development Center

Study of Navigation Channel Feasibility, Willapa Bay, Washington

Nicholas C. Kraus, editor

April 2000

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(in alphabetical order)

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Preface

This report describes a navigation channel reliability monitoring and evaluation study conducted at Willapa Bay, Washington, by the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL), Vicksburg, MS, for the U.S. Army Engineer District, Seattle (NWS). The study was established under a Partnering Agreement between the NWS and the Willapa Port Commission to investigate the feasibility of maintaining a reliable navigation channel through the entrance to Willapa Bay. Funding authority was provided by NWS to CHL on 6 April 1998, and a review draft of this report was submitted to NWS on 21 December 1998. Mr. Hiram T. Arden was the NWS point of contact for this study, with technical assistance and review of this report provided by Messrs. Arden, Eric E. Nelson, and Robert M. Parry, NWS.

The ERDC study team was under the technical direction of Dr. Nicholas C. Kraus, Coastal Sediments and Engineering Division (CSED), CHL, and task-area leaders were Dr. Adele Militello, Navigation and Harbors Division (NHD), CHL, for Long Waves, Sedimentation and Salinity; Dr. Jane M. Smith, Coastal Processes Branch (CPB), CSED, for Short Waves; Mr. William C. Seabergh, Harbors and Entrances Branch (HEB), NHD, for Engineering; Mr. Edward B. Hands, Coastal Evaluation and Design Branch (CEDB), CSED, for Morphology, Mr. Michael W. Tubman, Prototype Measurement and Analysis Branch (PMAB), CSED, for Field Data Collection Contract Management and Data Management; and Mr. Larry E. Parson, CEDB, CSED, for SHOALS Surveying.

Significant technical assistance in conducting the study was provided by Dr. S. Rao Vemulakonda, Estuaries, Bays and Lagoons Group, Tidal Hydraulics Branch (THB), Estuaries and Hydroscience Division (EHD), CHL, and Dr. Joon P. Rhee, PMAB, CSED. Additional technical support at CHL was provided by Ms. Mary Claire Allison, Dr. Andrew Morang, and Mr. Michael Martinez, CEDB, CSED, Mr. Fulton Carson, CPB, CSED, and Ms. Leonette J. Thomas, HEB, NHD. Secretarial and clerical support was provided by Ms. Janie G. Daughtry and Ms. J. Holley Messing, CEDB, CSED, and Ms. Myra E. Willis, HEB.

Assistance at NWS was provided by Mses. Elizabeth W. Bachtel and Joyce E. Rolstad at the NWS archive; Messrs. Denny S. Mahar and Lonnie M. Reid in surveying and cartographic control; and Mr. Thomas G. Landreth, captain of the survey vessel *Shoalhunter*. Field studies were conducted under contract with Evans-Hamilton, Inc. (EHI), led by Mr. Keith Kurrus, EHI. Portions of the field study analysis were subcontracted by EHI to Pacific

International Engineering[PLLC] (PIE). Mr. Harry Hosey and Dr. Vladimir Shepsis were the study leaders for PIE.

This study was conducted during the period April 1999 through June 1999 under the general supervision of Dr. James R. Houston, Director of CHL, respectively: Mr. Thomas W. Richardson, Chief of CSED; Mr. William H. McAnally, Chief of EHD; Mr. Bruce A. Ebersole, Chief of CPB; Ms. Joan Pope, Chief of CEDB; Mr. Robert T. McAdory, Chief of THB; Mr. Willam L. Preslan, Chief of PMAB; and Mr. Dennis G. Markle, Chief of HEB.

This study benefited from discussions and coordination meetings involving representatives from or staff members of the following organizations: Messrs. Gerald J. Heintz and James Neva, Port of Willapa; Mr. Randy Lewis, City of West Port; Mr. Jeff Cox, EHI; and Messrs. Hosey, Hugo Bermudez, Scott Fenical, David Simpson, and Nels Sultan, and Dr. Shepsis, PIE.

Contributors to this report are identified on the first page of each chapter. Dr. Kraus was the report technical editor. Mr. Markle was also assistant to Dr. Kraus and served as administrative point of contact for this study.

At the time of publication of this report, Dr. Lewis E. Link was Acting Director of ERDC, and COL Robin R. Cababa, EN, was Commander.

Conversion Factors Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
acres	4,046.873	square meters
cubic feet	0.028317	cubic meters
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
miles (U.S. nautical)	1.852	kilometers
miles (U.S. statute)	1.609347	kilometers
tons (long, 2,240 lb mass)	1,016.047	kilograms

1 Introduction[1]

This chapter presents the background of a study performed for the U.S. Army Engineer District, Seattle, to determine the technical feasibility of maintaining a reliable bar navigation channel into Willapa Bay, Washington. The study was authorized by the Seattle District in cooperation with the Port of Willapa Harbor under a Partnering Agreement. The study was conducted primarily by staff of the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory. Field data were collected by Evans-Hamilton, Inc. (EHI), and its subcontractor, Pacific International Engineering$^{\mathrm{PLLC}}$ (PIE), which is conducting independent field and analytical studies for the State Route 105 (SR-105) project at Willapa Bay (PIE 1997). Included in this chapter are overviews of the study site and discussions of related studies at Willapa Bay, study procedure, and scope of this report.

Background

Willapa Bay is a large estuarine system located on the southwest Washington coast, as shown in Figure 1-1. Its spring or diurnal range tidal prism, compiled from other sources by Jarrett (1976) at more than 10^{10} cu ft,[2] is one of the largest of all inlets on the coast of the continental United States. The magnitude of the tidal prism is produced by the broad bay area and relatively large tidal range at the site. The tidal range at the entrance to Willapa Bay, as measured by the National Ocean Service (NOS) of the National Oceanic and Atmospheric Administration, is approximately 7 ft. Daily wind speed is moderate, and river inflows do not contribute significantly to the flow through the entrance. Bay hydrodynamic processes are discussed further in Chapters 2 and 6.

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[1] Written by Dr. Nicholas C. Kraus, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS, and by Mr. Hiram T. Arden, U.S. Army Engineer District, Seattle, Seattle, WA.

[2] A table for converting non-SI units to SI units of measure is given on page xix.

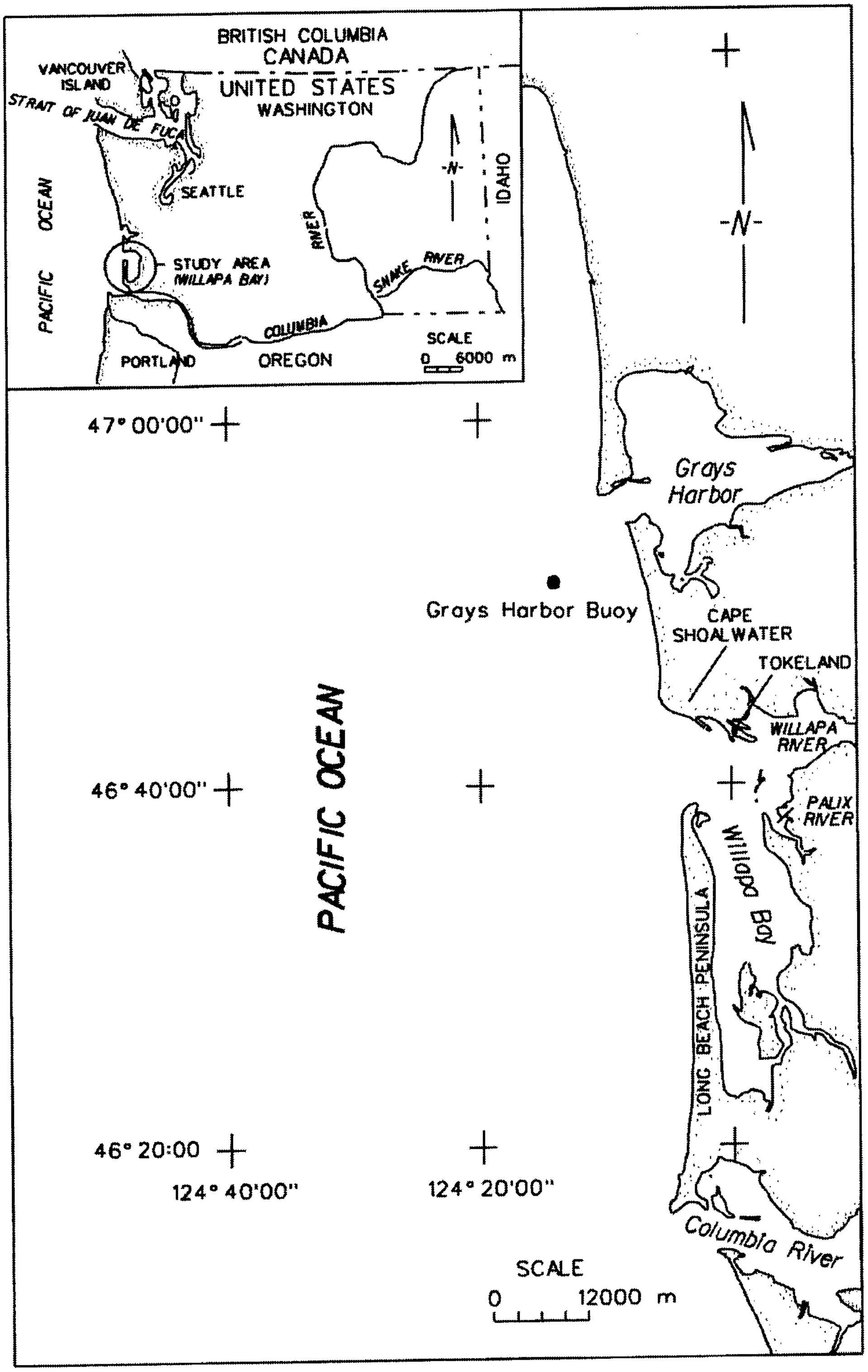

Figure 1-1. Regional location map for Willapa Bay, Washington

A wave hindcast covering 20 years (Jensen, Hubertz, and Payne 1990) predicted a mean significant wave height of 7.7 ft and a mean spectral period of 11 sec at hindcast Station 18 located in water depth of 33 ft in the vicinity of Willapa Bay. The largest wave in the hindcast had a height of 23.8 ft. Ruggierio et al. (1997) discusses the extreme waves and water levels along this coast during the 1997-1998 El Niño. Measurements made during the past 3 years at a nearshore wave buoy located south of Grays Harbor, as well as limited measurements of the waves at the entrance to Willapa Bay made in this study, confirm the magnitudes given by the hindcast. Properties of waves incident to the Willapa Bay entrance are discussed in Chapter 5, where numerical simulations of waves propagating into the bay are described. Wave measurements made in this study are described in Chapter 4.

The large tidal prism and energetic waves at Willapa Bay collectively act to transport millions of cubic yards of sediment on this predominantly sandy coast. The massive changes in shoals at the Willapa Bay entrance are discussed in Chapter 4. Neighboring bar entrances, such as Grays Harbor, the mouth of the Columbia River, and entrances along the coasts of Oregon and northern California are protected by jetties that are miles long. Construction of jetties improves navigation by stabilizing the position of the channel, focusing the tidal flow to clear sediment from the channel, and protecting vessels from waves as they transit through the surf zone.

The conception of a navigable channel at a wide and energetic inlet may seem improbable at first. However, information compiled in Chapter 3 shows that a natural but mobile channel some 25 ft deep typically penetrates the outer and middle entrance bars. The tidal prism at Willapa Bay maintains a dynamically stable channel cross section that usually contains a channel approaching design requirements. Inlet stability and bulk characteristics of the entrance, as well as details of the Federal navigation channel at the entrance, are discussed in Chapter 2.

Authorizations for a Federal navigation channel through the entrance to Willapa Bay are summarized in U.S. Army Engineer District, Seattle (1971), and in Chapter 2. The existing project was first adopted in 1916 and last modified through authorization in 1954. The authorization provides for a channel over the bar of the mouth of Willapa Bay to be 26 ft deep, measured to mean lower low water (mllw), and at least 500 ft wide. A bar channel of this dimension is required for existing shallow-draft commerce. Dredging of the deep-draft river channel of Willapa Harbor was discontinued by the Seattle District in 1976 because of inadequate benefits. Dredging for shallow draft continues at Willapa Harbor for facilities at such locations as Toke Point, Bay Center, and Nahcotta, shown in Figure 1-2. Since 1976, no maintenance dredging has been required along the Federal river channel leading up from Willapa Bay to port facilities located at Raymond, Washington.

The Washington Department of Transportation (WDOT) has recently constructed a groin and dike in the North (ebb) Channel at the entrance to Willapa Bay to protect SR-105. The adjacent shoreline (Cape Shoalwater) had been eroding at a rate of approximately 130 ft/year (Terich and Levenseller 1986; Komar 1998), and the erosion had endangered the State highway. The dike and groin are expected to significantly alter the flow and, possibly, the location and

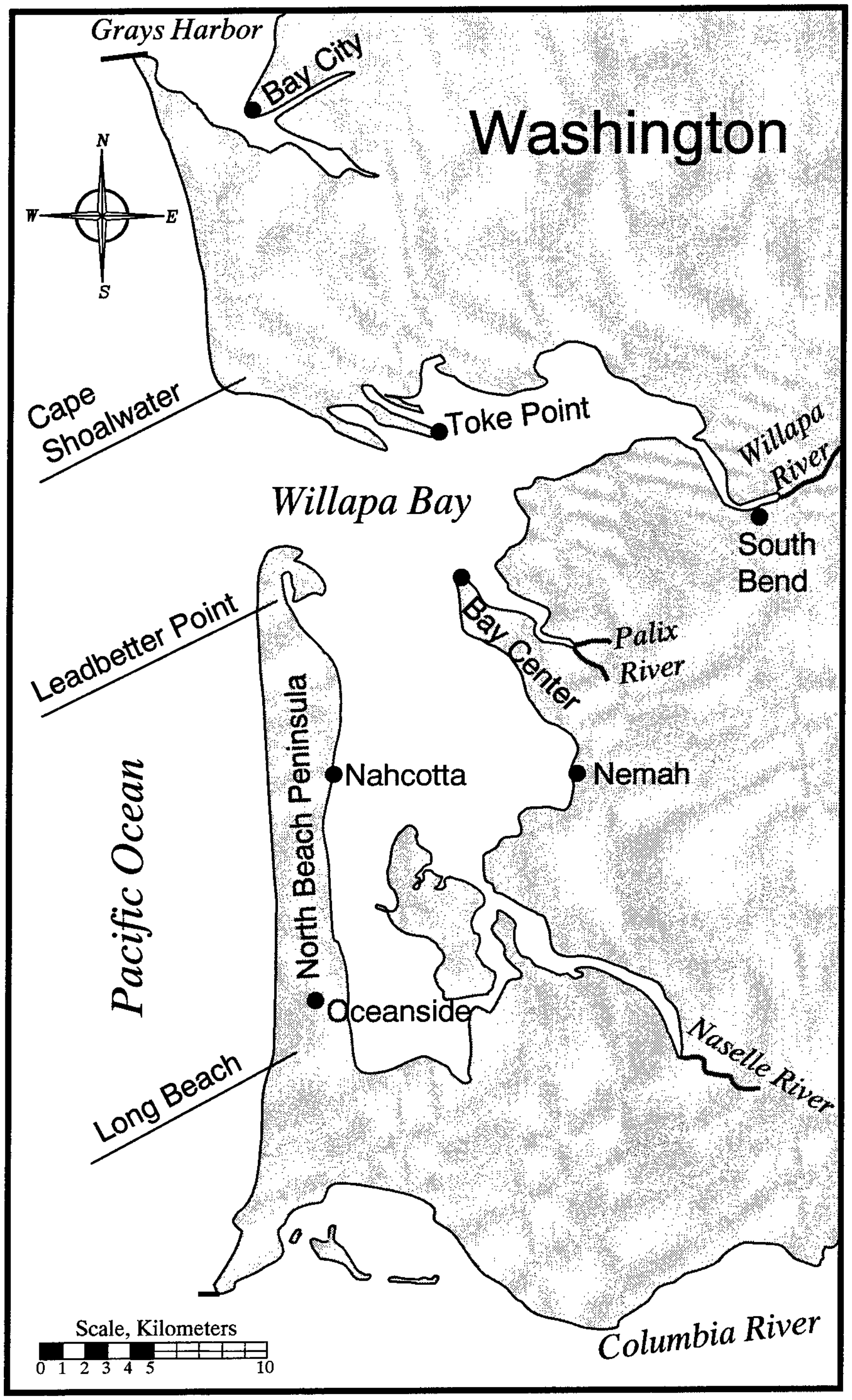

Figure 1-2. Willapa Bay and adjacent communities

stability of the North Channel. Action of the WDOT Cape Shoalwater shore protection project was considered in the channel stability analysis phase of the study.

Purpose of Study

The shifting channels at the entrance to Willapa Bay make bar navigation unreliable (U.S. Army Engineer District, Seattle 1971, 1995), and the local port cannot maintain or attract commercial users. Local interests have obtained Congressional support to determine if an economical channel can be established through the entrance bar. An economical channel implies a route that can be traversed safely under typical waves and tidal currents.

The Seattle District requested the U.S. Army Engineer Research and Development Center (ERDC) to conduct a study to determine the technical feasibility of maintaining a reliable channel (28-ft depth including advance dredging and overdredging allowance) over the entrance bar and into Willapa Bay. "Channel reliability" refers to stability of location and depth of the channel for an acceptable construction and maintenance cost, as well as hydrodynamic conditions for safe passage.

Ebb currents exiting the southern arm of Willapa Bay (the arm extending toward Oceanside) are directed toward the landmass of Cape Shoalwater, where they turn and run west in a relatively deep North Channel. Water exiting the tidal flats along the Willapa River also tends to flow out of the North Channel. Other channels through the bar exist ephemerally, including a Middle Channel and a South Channel, sometimes called the Leadbetter Channel in the literature. The typical locations of these channels are shown schematically in Figure 1-3. Multiple bar channels through the entrance sometimes exist, but typically one channel dominates. Properties of the natural channels, including location, persistence, and depth, are discussed in Chapter 3, based on an extensive record of bathymetry surveys spanning more than a century. The presence of these channels and their possible exploitation as a navigation channel form the basis for developing channel design alternatives, as described in Chapter 2.

Related Studies

The State of Washington is conducting two studies of coastal and inlet processes of interest to the present proposed effort. The Washington Department of Ecology (WDOE), in a joint study with the U.S. Geological Survey, is making a regional coastal assessment that includes analysis of all available historic and present data on shoreline position and bathymetry (Gelfenbaum et al. 1997; Kaminsky et al. 1999).

The WDOT, through its lead contractor, PIE, and PIE subcontractor, EHI, is collecting data on waves and currents at the entrance and is monitoring coastal and inlet processes for the SR-105 Cape Shoalwater shore protection project (PIE 1997). PIE has also conducted a morphological analysis of bar channel migration, based on earlier work by the Seattle District and others. The NOS maintains a long-term water-level station in Willapa Bay at Toke Point. As much as possible within study constraints, all relevant data were considered and joint

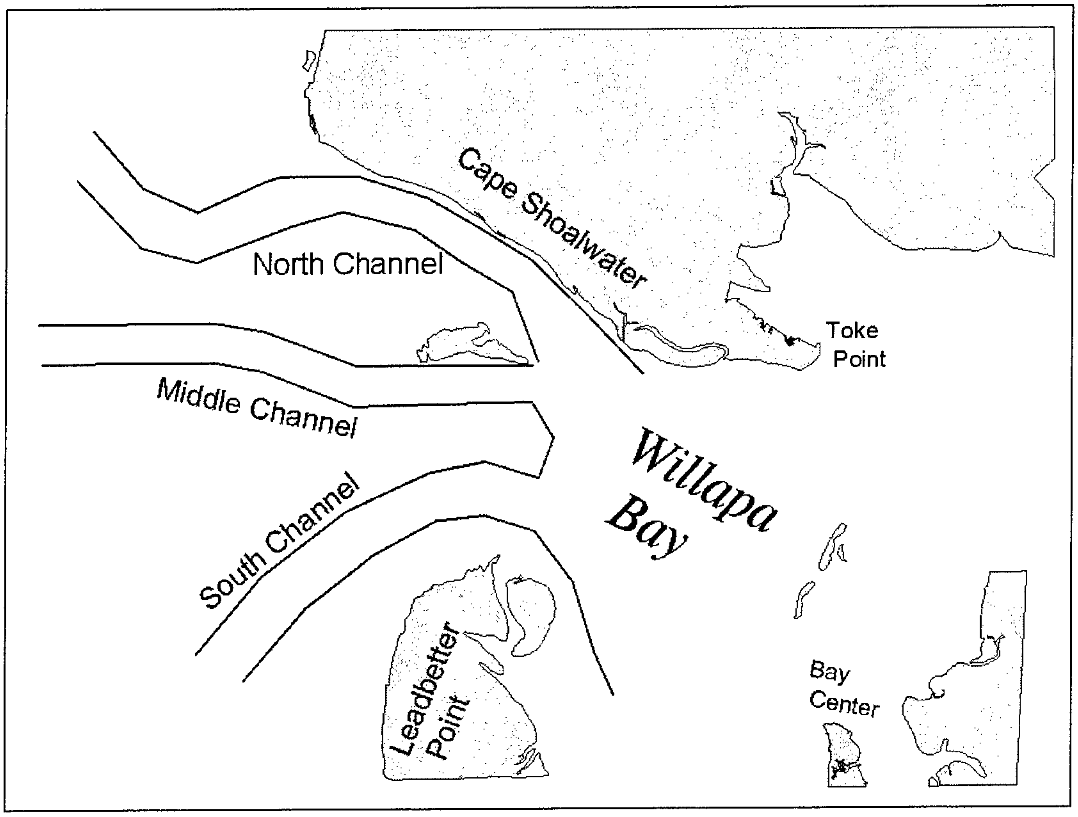

Figure 1-3. General locations and orientations of natural entrance channels to Willapa Bay

and coordinated efforts made with these agencies and organizations, including the University of Washington, for efficient and cost-effective conduct of this study.

Study Procedure

The study required focused efforts of several specialists who participated as a team in developing approaches and procedures and in conducting the required work. Meetings and briefings were held at the study site and at the Seattle District, as well as at ERDC, with participation from the Port of Willapa, WDOE, PIE (representing local interests), EHI, Seattle District, and ERDC. ERDC investigators also toured the study site, made reconnaissance trips for locating instruments in and around Willapa Bay, and participated in data collection.

The study was developed as a simultaneous effort in two major tasks. One task involved analytical and numerical studies covering the following:

a. Engineering activities in consideration of entrance channel alternatives and their relation to maintenance and operation of a navigable channel.

b. Formulation of alternative designs for a reliable bar navigation channel.

c. Analysis and interpretation of inlet morphology change.

d. Numerical modeling and associated analyses of wave transformation at the entrance, circulation and water level throughout the bay, transport of sediment at the entrance, and salinity distribution in the bay.

The other task involved sustained data collection and analysis in support of the analytical and numerical efforts, as well as documentation of the background condition of waves, wind, current, and morphology at the entrance, and water level and salinity around the perimeter of the bay. A survey of the discharge and salinity within the bay was made by two boats. Bathymetry data, the most basic information upon which most of the study components depend, were collected by the Corps' airborne laser system and by a considerable dedicated effort of the Seattle District's main survey boat, the *Shoalhunter*.

The Seattle District requested preliminary study results by 1 February 1999, which required rapid planning and mobilization to deploy instruments in the severe environment of the entrance to Willapa Bay and along the substantial perimeter of the bay. Authorization to proceed was received in April 1998, and a data collection plan was developed and instruments procured and leased for gradual deployment commencing in July 1998. Data were downloaded in a series of instrument servicing trips and subsequently quality checked and reduced expediently for distribution to the analytical and numerical components of the study. The difficult but successful field data collection program is described in Chapter 4.

An interim copy of this report was transmitted to the Seattle District on 21 December 1998 for review and discussion by the District and local interests. Further study was then conducted, leading to this final report.

Scope of Report

This report documents the procedures, results, and conclusions of the subject study. Study team members described their work in individual chapters. The chapters were planned to form a coherent approach in meeting the study objective of determining the feasibility of a reliable bar navigation channel into Willapa Bay. The approach and content of all chapters were coordinated, and an attempt was made to provide sufficient background information and cross-referencing to allow each chapter to stand alone with regard to its particular subject matter.

Chapter 2 documents the Federal navigation project at Willapa Bay and describes approaches to investigating channel reliability in a natural inlet. Chapter 3 presents results of a morphology analysis based primarily on surveys of the bathymetry spanning more than a century. Chapter 4 summarizes the field data collection program and general properties of the data sets. Chapter 5 presents results of short wave numerical simulations. Chapter 6 presents analysis of tidal circulation, sediment transport, and salinity change. Chapter 7 discusses the channel alternative designs considered through the report. Appendices A-H contain supplementary and background information.

Units of Measurement

Dimensions and quantities originally reported in American Customary (non-SI) units on engineering documents and in the literature are retained. A table of conversion factors from non-SI to SI units is given on page xix. Oceanographic and meteorologic measurements and calculations, such as of waves, water current, and wind speed are given in SI units.

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2 Entrance Engineering[1]

This chapter describes the existing navigation project and reviews maintenance dredging that was conducted in the past. Vessels transiting the navigation channel at Willapa Bay are discussed, and a project design vessel for this study is defined. The limits of environmental operating conditions for vessels are reviewed. An examination of channel stability is made, and significant tidal inlet parameters are presented to aid in considering a safe and reliable Willapa Bar navigation channel. Placement or disposal sites of dredged material are also discussed, together with estimates of the associated costs. Further information on disposal sites is given in Appendix H.

Project

This section summarizes the navigation project at the entrance to Willapa Bay. Depths are referenced to mean lower low water (mllw). Figure 2-1 shows the project layout. The seaward channel over the bar does not appear in this figure because it varies in position in time and is marked by the U.S. Coast Guard (USCG) based on U.S. Army Engineer District, Seattle, surveys. The existing project, adopted 27 July 1916, and last modified 3 September 1954 (U.S. Congress 1916, 1954), provides the following:

a. A channel over the bar at the mouth of Willapa Bay, 26 ft deep and at least 500 ft wide. Overdepth of 2 ft is allowable.

b. A channel 24 ft deep and 200 ft wide from deep water in Willapa Bay to the foot of Ferry Street in South Bend, then 300 ft wide to the westerly end of the narrows, then 250 ft wide to the forks of the river at Raymond, including a cutoff channel 3,100 ft long at the Narrows.

c. A channel 24 ft deep and 150 ft wide up the South Fork to the deep basin above Cram Lumber Mill, and up to the North Fork to 12th Street, with a turning basin 250 ft wide, 350 ft long, and 24 ft deep.

d. A channel 10 ft deep and 60 ft wide from deep water in Palix River to Bay Center Dock.

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[1] Written by Mr. William C. Seabergh, Dr. Nicholas C. Kraus, and Mr. Edward B. Hands, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS; Dr. Vladimir Shepsis, Pacific International Engineering^{PLLC}; and Mr. Hiram T. Arden, U.S. Army Engineer District, Seattle, Seattle, WA.

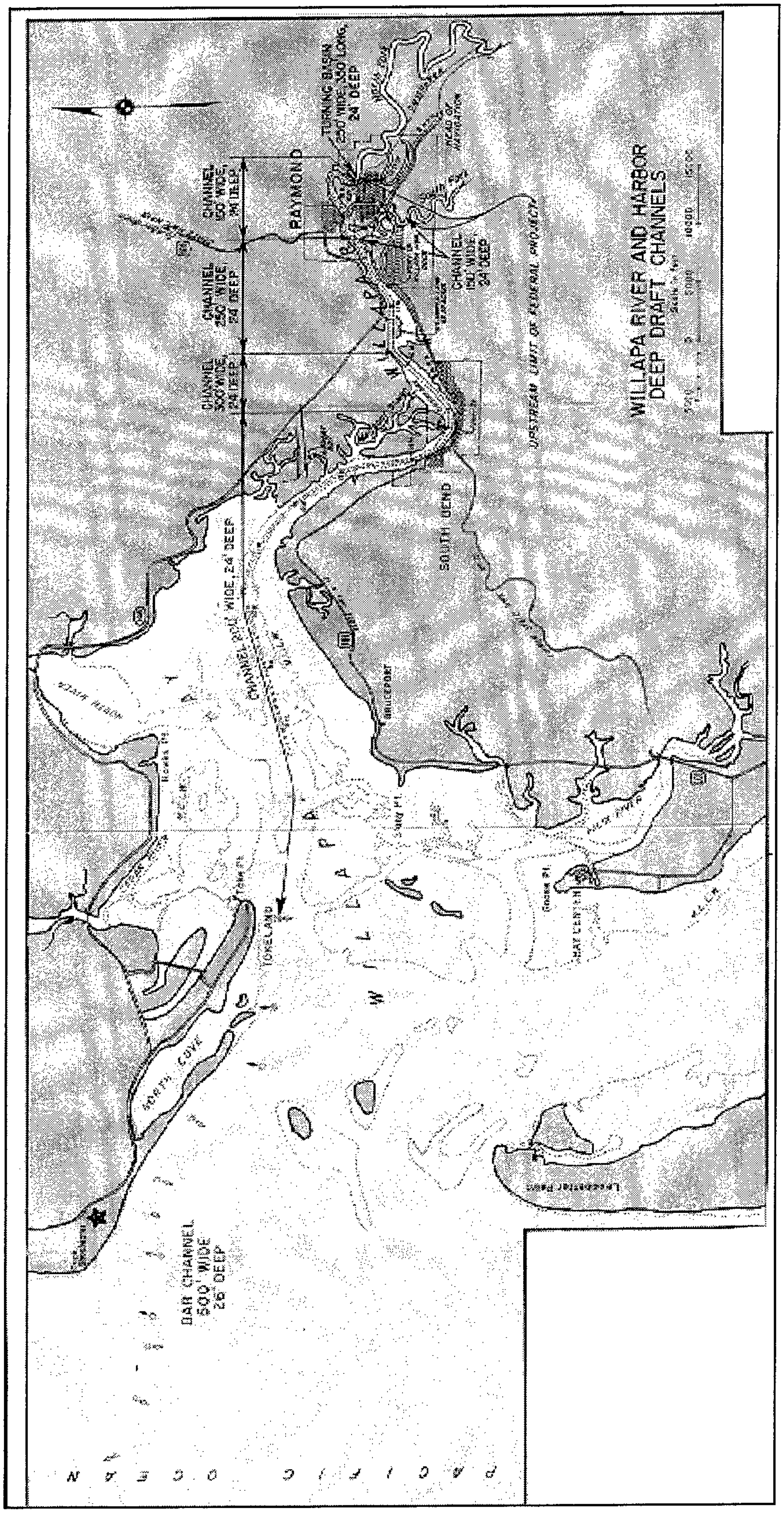

Figure 2-1. Willapa River and Harbor navigation channels (project map, Seattle District)

e. An entrance channel 15 ft deep and 100 ft wide and a mooring basin 15 ft deep, 340 ft wide, and 540 ft long adjacent to port wharf at Tokeland.

f. An entrance channel at Nahcotta 10 ft deep and 200 ft wide and a mooring basin 10 ft deep, 500 ft wide and 1,150 ft long, protected by a rubble-mound breakwater approximately 1,600 ft long.

The entrance channel and Willapa River channels were completed in 1936, with additional widening completed by 1958. Annual maintenance costs for 1964-1970 averaged $468,400, but did not provide full project depth year-round. At the 1998 time of study initiation, the controlling depth over the curved north channel ebb shoal was about 20 ft, and the controlling depth in the river channel reach was about 17 ft. (Note: By October 1999, the controlling depth over a straight-out north channel improved to about 24 ft.) As can be noted from the authorized widths and depths of the river channel reach, a variation existed in the size of vessels capable of traversing certain parts of the project.

Recent dredging amounts taken from the bar channel are shown in Table 2-1. Over a 23-year period, the annual volume removed averaged 288,000 cu yd, with a maximum of 610,000 cu yd dredged in 1969, an indication of the cyclic movement of large masses of sediment. One should also note that typically the authorized depths were maintained for only a few months before depths again became less than 26 ft, indicating substantial sediment infiltration along the channels. Dredged volumes were determined in part by the schedule of availability of Government hopper dredge equipment. A complete chronology of Willapa Harbor dredging is presented in Table H-1 of Appendix H.

Table 2-1
Willapa Harbor-Bar Channel O&M Hopper Dredging

Year	Volume 1,000 cu yd	Year	Volume 1,000 cu yd
1951	308	1965	955
1953	30	1966	303
1956	187	1967	341
1957	256	1968	459
1958	253	1969	610
1959	237	1970	324
1960	222	1971	340
1961	282	1972	274
1962	453	1973	189
1963	282	1974	42
1964	278	1997 Test Dredging	80

Note: dredged volumes reflect schedule of availability of government hopper dredge, i.e., dredging may not have returned the channel to design dimensions.

Vessels

Historic log and lumber ships planning to enter Willapa Harbor typically would have fully-loaded drafts of 28 to 33 ft. Because of the inadequate channel depths and shifting outer bar channel, the Twin Harbor Pilots Association has

restricted loadings to 20-ft drafts, or one-third capacity, which require the vessel to top off at another port. Also, the pilots have restricted sailings to daylight hours. A table in the Feasibility Report (U.S. Army Engineer District, Seattle, 1971) for years 1964-70 indicated that 1964 had the highest number of log exports and lumber shipments, with 88 shipments. The number decreased yearly to 45 shipments in 1970. This industry has migrated to Grays Harbor, so it is questionable if the entrance should be designed for this size vessel. In addition, the newer lumber ships have 37-ft drafts, which would require a deeper authorized channel. Most likely though, if Willapa Bay is to be made a viable port again, some type of cargo vessel should be considered in the design process.

Commercial fishing vessels, charter boats, towboats, and recreational and other small craft presently would have little difficulty with the previously maintained bar channel depths (U.S. Army Engineer District, Seattle, 1971). Drafts of these vessels are on the order of 12 to 16 ft. Potential harbor tenants originate from the tuna boat fleet, and these vessels have 17-ft drafts. Channel shifting, strong currents, and turbulence created by storm waves breaking over shoals make passage hazardous for this size vessel. No criterion was found to determine the level at which the wave breaking turbulence becomes critical to this size vessel.

Tugs (draft, 13 ft) and their barges (draft, 11-12 ft, 236-ft-long by 60-ft-wide) require straight channels for accommodating the length of the towline. Grounding incidents point to channel curvature as a main cause of navigation difficulty. The tugs require waves less than 9 ft for safe navigation. Barges experience 15 to 20 ft of vertical motion around still-water level in the presence of such waves, based on 30-deg roll with 60-ft beam and 10-deg pitch with a 236-ft length.[1] The vertical motion contributes to produce an effective draft of 27 to 32 ft mllw. If vessels enter and exit at the +6-ft tide level, a channel depth of 21 to 26 ft is required. Pitching dominates at the outer bar and, at the inner bar, rolling is likely to dominate from reforming waves.

Design Vessel

The design vessel is "usually ...the largest vessel of the major commodity movers" (Headquarters, U.S. Army Corps of Engineers 1996). Another approach, taken in the Grays Harbor project, is to select a vessel that carries the majority of product tonnage, then check for adequacy of movement of the largest vessels. The design ship draft for Grays Harbor was 37 ft (for a 46-ft-deep by 1,000-ft-wide authorized project). Presently, the decision for Willapa Bay would have to be based on tug and barge traffic to satisfy the criterion of the largest vessel of the major commodity mover. (This decision follows because the deeper draft lumber ships are not calling at Willapa Bay. However, tugs and barges are also not calling there at present, but they are the most recent commodity hauler other than commercial fishing vessels.) For the given project authorization of 26 ft, the tug-barge combination or a commercial fishing vessel apparently should be the design vessel. To design for the draft of the most recent lumber ships would require a significant increase in authorized depth.

[1] Letter to U.S. Army Engineer District, Seattle, from Brix Maritime Company, 12 May 1992.

Design Criteria

Criteria for determining entrance channel depth, width, and alignment (with respect to wave approach) depend upon the specific vessel. Figure 2-2 shows the factors involved in determining channel depth. Taking Figure 2-2 as a guide and assuming a 13-ft draft for a tug, combined squat and trim is less than 2 ft for this vessel traveling at 10 knots in a wide fairway such as the ocean bar region (Headquarters, U.S Army Corps of Engineers, in preparation). Advance maintenance dredging and dredging tolerance (neither is included as part of the design (authorized) depth) will be assumed combined as 2 ft. Safety clearance is normally taken as 2 ft.

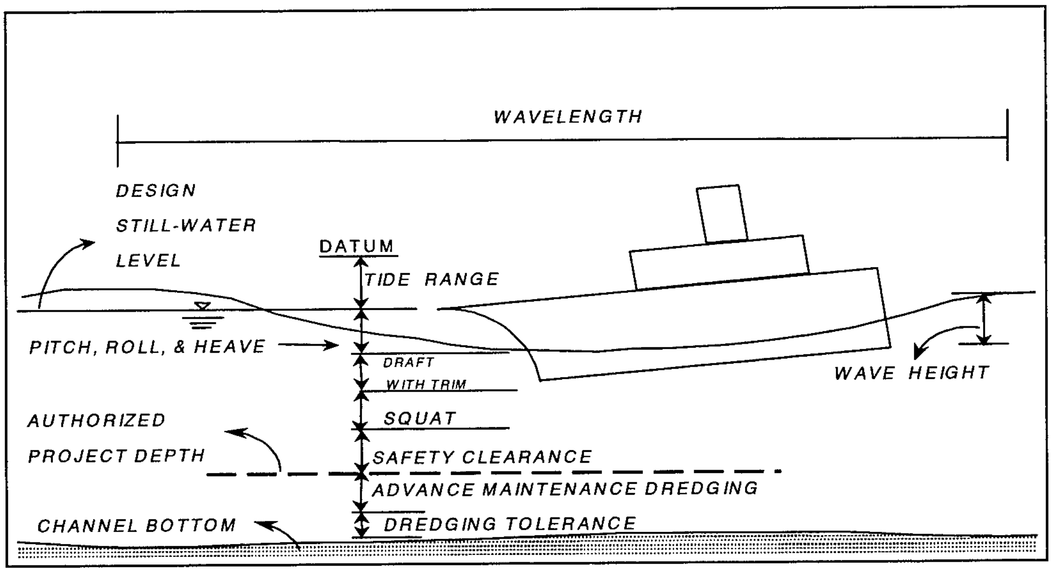

Figure 2-2. Channel depth allowances and wave steepness defined as wave height divided by wavelength

At a high-wave location such as Willapa Bay, a large part of the channel design depth is associated with the vessel movement because of vertical components of pitch, roll, and heave resulting from wave motion. Wave-accompanying effects tend to increase as wave height increases and decrease with greater ship length. A typical ship motion response amplitude operator (RAO) is 1.2. The RAO is a dimensionless factor with which to multiply wave amplitude (amplitude being one-half the wave height) for determining the distance below the still-water level to which the ship will move in waves of given height.

Values of RAO measured in the mouth of the Columbia River ranged from 0.9 to 2.2 for bulk carriers, 1.0 to 2.1 for tankers, and from 0.4 to 1.2 for container ships (Headquarters, U.S. Army Corps of Engineers, in preparation). If an operational wave height of relatively high energy is taken as 9 ft, then the additional channel depth required would be 1.2×4.5 ft = 5.4 ft.

Combining wave amplification response and draft, squat, and safety clearance determined previously yields a value of 22.4 ft. This depth contains tolerance and leaves room for navigating in higher wave conditions, up to 15-ft

wave heights in the channel. The value for a fishing vessel with a 17-ft draft would be 26.6 ft for the 9-ft wave condition, probably an acceptable value because of inclusion of a safety clearance. In other words, wave conditions of 9 ft or less in the channel would be necessary for relatively safe operations for the design vessels.

Required channel width can be determined by evaluating factors as outlined in Permanent International Association of Navigational Congresses (1997). The basic maneuvering lane for a vessel of low maneuverability (such as a tug/barge system) is 1.8*B*, with *B* the beam of the design vessel. For strong crosswinds of a slow-moving vessel, a factor of 1.0*B* is added. If strong crosscurrents are present, 1.3*B* is added. For strong channel currents and slow vessel speed, 0.4*B* is added. For significant wave heights greater than 10 ft, 1.5*B* is added. If visibility is frequently poor, one should add 0.5*B*. For shallow, hard bottoms, one should add 0.3*B*. The total is 6.8*B*. For a 60-ft beam, a width of 408 ft would be required. The 500-ft authorized width meets that requirement. A possible reason and benefit for increasing channel width is to add storage capacity for sediment moving into the side of the channel from spits and shoals. The storage capacity would extend the time interval between dredging operations required to maintain the authorized channel depth and width, but would require a dredge to remain on site longer during maintenance.

At times an S-curve is present as a spit emerging from the north beach pushes the North Channel to the south before it can again turn to the west and exit to deeper water. Chapter 3 discusses this channel pattern in its historical context. In this study, 1,500 ft of channel width was estimated to be required for tows entering an S-curve, allowing for as much as a 45-deg angle off the center line of the tug. Towlines are typically 300 to 500 ft long, with a minimum of 100 ft. Tugs are typically in the 65- to 100-ft-length range, and barges are 150 to 400 ft long. The 1,500-ft width is based on the maximum dimensions of the towline, tug, and barge.

The orientation of channel alignment with respect to wave approach (and thus vessel alignment to some degree; however, the vessel may change its orientation with respect to waves if the channel is wide enough) enters into consideration of navigability. In examination of wave conditions for various alternatives, a value of ±45 deg was selected based on limited output of a ship motion model called HYDRO.[1] Figure 2-3 shows the effect of wave angle on a bulk carrier in terms of wave period versus the RAO. The three wave angles examined were 0 deg (the vessel is moving directly into the wave crest), 45 deg to the vessel center line, and 90 deg, with the wave hitting the vessel broadside. Important to note is that the 0- and 45-deg curves are in about the same range RAO over the various wave periods. For the 90-deg wave, the RAO peaks up to a much higher value, as the vessel is near resonance with the 12-sec wave in the roll mode. Based on this example, the ±45-deg window was selected for evaluating alternatives.

[1] Personal Communication, 1998, Mr. Frank Sargent, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS.

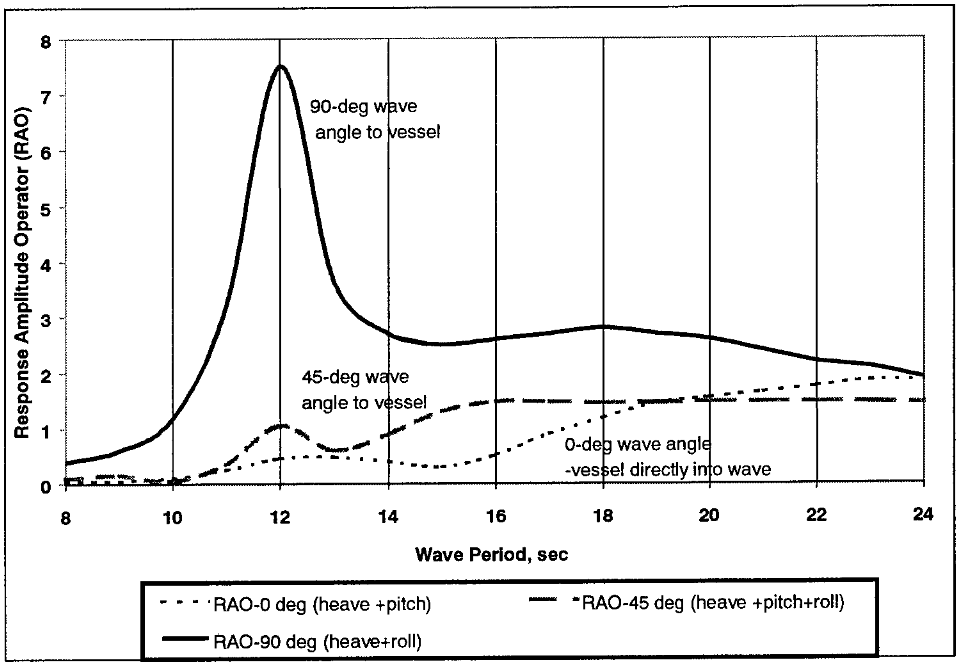

Figure 2-3. Response amplitude operator versus wave period for 0-, 45-, and 90-deg wave angles approaching an 824-ft-long, 106-ft-beam, 42-ft-draft bulk carrier

Kraus (1997) proposed a wave steepness criterion for establishing navigation safety in a channel. Wave steepness is defined as the wave height divided by the wavelength. A critical value of 0.05 was recommended as preliminary guidance, above which there was concern for the safety of vessels with lengths on the order of that of the incident waves. The incident wave steepness increases if waves meet an opposing (ebb) current. The steepening of individual waves in a wave train creates a "washboard" or chop that can be difficult to navigate.

Grays Harbor Experience

Grays Harbor, Washington, lies to the north of the Willapa entrance and is a Federal navigation project. Information from the Grays Harbor General Design Memorandum (U.S. Army Engineer District, Seattle 1989) indicates that recent log vessels are 25,000 to 30,000 dwt, with 570-ft lengths, 87-ft beam, and design drafts of 34 to 37 ft, departing at higher tide levels. At Grays Harbor, the outer bar channel is aligned along a southwest azimuth based on pilot preference. Grays Harbor pilots indicate they are concerned about being set by currents, and by the wind and vessel pitch, rather than by roll of the vessel. The channel dimensions for the Grays Harbor project are 30-ft depth by 600-ft width over the bar and through the entrance.

Operationally, transits from docks begin 3 hr or more after low tide to move against the flood current for better steerage, to obtain additional keel clearance, and to avoid hazardous ebb-flow conditions. The 30- to 37-ft draft vessels transit the entrance channel only in optimum conditions. Depth allowances for vessel pitch, roll, and heave in waves moving over the ebb bar are 14 ft. Channel width

for strong currents and waves is 600 ft in the entrance and 1,000 ft at the seaward end of the entrance channel and across the bar.

The maximum significant wave height allowable for outbound transits is about 8 ft for flood tide conditions. Studies by Wang et al. (1980) showed maximum excursion of the bow will be less than 14 ft for 95 percent of the time under these conditions. Maintenance dredging at Grays Harbor occurs mostly in the interior. The twin jetties at Grays Harbor function to maintain depths in the entrance channel and over the ebb shoal so that no significant maintenance dredging is required for a 30-ft-depth channel (U.S. Army Engineer District, Seattle, 1989). The presently-maintained project (46-ft-deep channel 1,000 ft wide at the entrance bar) is estimated to require dredging of 100,000 to 200,000 cu yd/year in the entrance channel and less than 100,000 cu yd/year at the entrance bar.

Existing Navigation Conditions and Navigation Safety[1]

Vessels avoid exiting Willapa Bay on the ebb because of longer transit time when waves are present. When a vessel exits on the ebb, with some waves breaking in the channel, reading the waves and estimating which ones will break is a difficult procedure for the navigator. The navigator needs to slow down as the ebb current pushes the vessel ahead while he or she evaluates if a wave will break. The vessel is attempting to avoid the chaotic wave action of the breaking wave that would then wash over it. For example, every 5 min, two or three waves may break. This lower exit speed increases time of exposure in the entrance channel. In the case of tug-barges, the decrease in speed reduces the control of the barge, permitting crosscurrents and wind to push the barges out of line with the tug.

In the region from Cape Shoalwater into the bay, strong chop exists that can be difficult to navigate and, in the case of tows, again makes the barge wander behind the tug. Also, while in this region, no deck work can be performed, but the vessel is not in danger similar to that when on the bar channel.

When a vessel is traveling in the S-curve of the North Channel, extreme breaking conditions may exist for waves from the northwest and the west and wave angle with respect to the vessel travel may be large. The washboard conditions are difficult for navigation. Within the seaward east-west part of the S-curve, 400 ft of migration of the channel thalweg can occur within one month. Conditions are usually acceptable if swell height is less than 3 ft and the vessel travels in late ebb, flood, and early ebb. Vessels will avoid moderate and greater ebb-flow conditions when moderate to large waves are present. No knowledgeable local boat captain wants to navigate the bar channel at night with ebb conditions. Navigation at slack or early flood is acceptable if there is not much swell; otherwise, boat captains will wait for stronger flood flows before exiting during conditions with larger waves.

[1] Information in this section was obtained during a 30 October 1998 telephone conversation with Mr. Randy Lewis, formerly Warrant Officer USCG (retired) stationed at Willapa Bay.

Willapa Bay Design Conditions

Based on available information, two design vessels were selected: a tuna fishing vessel with a draft of 17 ft and a tug/barge vessel combination (13-ft draft tug and barge with a 12-ft draft, 236 ft long by 60 ft wide). The best alternative should be selected in part considering the highest percentage of time with waves less than 9 ft in height on a flood tide. Wave angle with respect to channel alignment should be less than 45 deg to avoid significant roll of the vessel. Head-on waves do not always present the best wave angle for tug-barge combinations because this can produce extra tension in the towlines if the tug-barge systems are in phase with one another. Channel depth must be greater than 26 ft at all times. The barge-tug vessel will require a relatively straight channel alignment and a minimum width of 1,500 ft if an S-curve exists.

Inlet Channel Stability

Willapa Bay has existed as a relatively stable inlet for more than a century by maintaining a strong tidal exchange of water entering and exiting the inlet. A Coast and Geodetic Survey nautical chart, Number 6185, for the period of 1887-1897, indicated that the mean tide range was 7.4 ft at Sealand (Nahcotta) and 7.5 ft at South Bend (with a mean ocean tide range of 6.2 ft). Present National Ocean and Atmospheric Administration Tide Tables indicate that the mean range is 7.9 ft at Nahcotta and 7.8 ft at South Bend. These numbers may not be directly comparable because of improvements in measurement technology, but in each case, one can note the recent bay range was only slightly greater than the earlier tide range. Consequently, the overall tidal hydraulic condition of the inlet system has been relatively constant over the past one hundred years.

If the slight increase in bay tidal range has actually occurred, this would follow from a trend seen in the measurement of minimum cross-sectional areas of the inlet entrance. The minimum area (determined by measurement of the inlet cross section at the minimum width, considered to be a reasonable assumption) increased from 450,000 sq ft in 1877, to 480,000 sq ft in 1937, to 520,000 sq ft in 1967, to 530,000 sq ft in 1996. There is some amount of uncertainty in these area calculations because of lack of data in very shallow regions together with the usual possibility of measurement error related to many other factors. Errors in tidal datums could lead to large errors because of the large width of the inlet. For example, a variation of ±0.5 ft over the approximate 20,000-ft width leads to a cross-sectional variation of ±10,000 sq ft. Therefore, a variation of 20,000 sq ft should be assigned.

The minimum cross-sectional area is considered as a measure of the tidal prism (defined as the volume of water entering (or leaving) the bay during a flood (or ebb) portion of the tidal cycle). The minimum cross-sectional area A_c (measured at mean tide level) is related to the spring or diurnal tidal prism P as

$$P = 5 \times 10^4 A_c \tag{2-1}$$

This equation is from O'Brien (1931) based primarily on data from Pacific Ocean inlets of the United States. Jarrett (1976) determined similar relationships for

east coast, west coast, and Gulf coast inlets. Using Equation 2-1 and the 1996 area, an estimate for P would be 2.65 10^{10} cu ft, a value near the maximum for United States inlets. Escoffier (1940, 1977) developed an analytical or graphical method to examine the stability of coastal inlets. This technique (Seabergh and Kraus 1997) is applied to Willapa Bay in Figure 2-4.

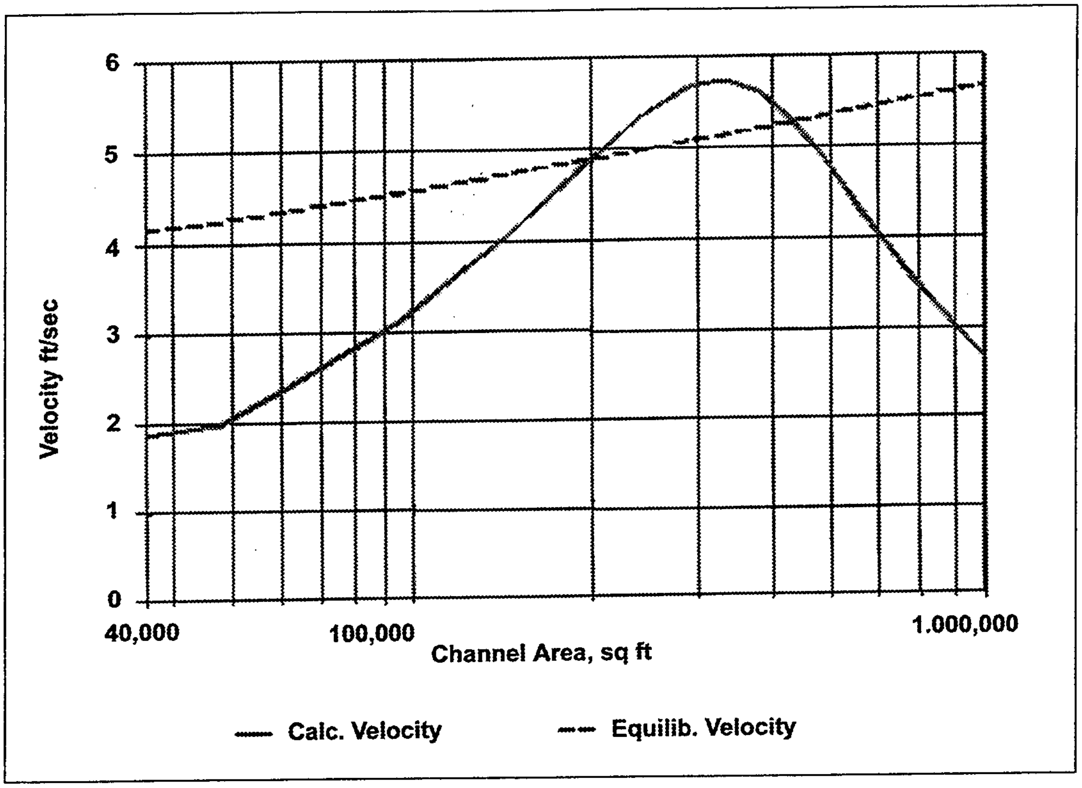

Figure 2-4. Escoffier diagram for Willapa Bay. Plot shows average maximum velocity versus minimum inlet area (solid line) and the equilibrium velocity curve (dashed line). Intersection of curves on right side indicates stable equilibrium area for Willapa Bay, 447,000 sq ft, based on assumptions discussed in text

The intersection of the curves on the right side in Figure 2-4 is a stable equilibrium area (447,000 sq ft). The inlet area would vary about this value through sediment flux into and out of the channels and changing tidal conditions (e.g., spring to neap tide range variation). The left-hand intersection is an unstable equilibrium area (200,000 sq ft). If the minimum area became smaller than this minimum value because of a large influx of sand, the inlet would close. The historical values are on the high side of stable equilibrium. These higher values of inlet area are in qualitative agreement with the long-term scouring of the north shore. This analysis shows that overall the inlet is stable at the minimum cross-sectional area location and, by flow continuity, the larger offshore cross sections maintain relatively constant flow areas even as the distribution of flow and number of channels change (Chapter 3). However, channel migration caused by movement of sand in the more seaward region creates local instability of channel location and hazards to navigation because of

movement and curvature. These patterns of shoaling and channel movement are governed by the influx of sediment discussed in Chapter 3.

The result of a reduction in channel flow area can be determined by consulting Figure 2-4. For example, building the State Route (SR)105 Emergency Project on the north shore reduced the flow area in the channel 30,000 sq ft. Assuming the reduction is near the location of the minimum cross-section area, the flow area (530,000 – 30,000 sq ft) would still be above the stable equilibrium area (447,000 sq ft). Even if the inlet were at its stable area of 447,000 sq ft, a reduction of 30,000 sq ft in inlet area to 417,000 sq ft would still be in the range where the inlet area would be returned to its stable equilibrium area by cutting a deeper or wider channel. Bathymetric data from recent surveys support this conclusion.

An increase in flow area, as produced by excavation of a larger channel, would move along the solid curve to the right showing an increase in area, but a reduction in velocity. The result would be an infilling of the channel and a return to the equilibrium area. This evaluation describes the system as a whole and does not link changes in one region to changes in another; e.g., if the South Channel would shoal, one would expect an increase in area in the Middle or North Channel to maintain the equilibrium of the whole system, but this analysis cannot predict which channel would deepen.

Development of Design Alternatives

The development of alternatives is based on selection of channel alignments and channel size that meet certain criteria. First, a reliable channel is desired. A reliable channel is one that can be maintained to provide safe navigation based on present and design vessel usage. It must also meet a certain benefit cost ratio and be environmentally acceptable (not discussed in this report). Therefore, the three engineering points to consider in evaluating alternatives are as follows:

a. Estimated project scope (initial dredged volume).

b. Estimated average annual maintenance volume.

c. Navigation safety.

Based on examination of historical maps of the inlet as discussed in Chapter 3, three main locations of potential egress to the ocean from Willapa Bay exist, not all of which are usually present at the same time (Figure 3-1). A North Channel is identifiable on all maps of the inlet; however, because of spit growth from the north, sometimes the North Channel has exited into the ocean through a wide range of angles, from toward the northwest to the southwest. The present Middle Channel typically does not connect directly to the ocean bar, but is incised centrally in the middle of the inlet. Its present geographic location was once the main exit to the sea, probably because of its alignment with the Willapa River. During the past 100 years with the movement of the channel northward and erosion of Cape Shoalwater, the Middle Channel has not been the dominant flow channel. The South Channel is ephemeral, and it is incised in the shallow shoal similar to the Middle Channel. The analysis in Chapter 3 characterizes the longevity, location, depths, and orientation of the channels in a historical perspective.

Possible alternatives are summarized in Table 2-2, and the initial dredging volumes required (based on 1998 bathymetry) are presented in Table 2-3. During the 18-month period of Phase I studies, the volume of sediments associated with the bar channel improvement from a 20-ft deep curved (Alternative 3B) alignment to a 24-ft deep (approximating Alternative 3A) alignment exceeds 20 million cu yd without dredging. Volumes vary between 0.2 and 9.9 million cu yd for the depths and widths described in Table 2-2. The list of 19 alternatives was developed without preconception of a preferred alternative, and moderate to extreme (expensive) alternatives were considered. The alternatives were screened in the course of the study, with some initially deleted based on reasoning described in the next paragraphs, and some alternatives deleted after reconnaissance numerical simulations as described in Chapter 6, to arrive at a subset for detailed examination. An additional aspect in evaluation of alternatives is potential beneficial uses of dredged material, such as for protection of the North Beach shore and creation of bird islands, oyster habitat, and wetlands (Appendix H). Further refinement of the described alternatives and possible addition of new alternatives are expected during the future course of this study.

Table 2-2 shows that the existing procedure (numbered as Alternative 1) is considered an alternative as it is the least costly approach, and it also defines the existing condition against which other alternatives may be compared. Alternative 2 is considered in terms of a future existing condition where opportunistic dredging may provide a reliable channel. This alternative is defined by future conditions and thus is not examined in the modeling. All Alternative 3 variations focus on using the North Channel. Alternative 3A proposes a straight channel at its present northernmost location (Figure 2-5). Alternatives 3B and 3C are based on improving the width of the S-curve, which evolves if the spit from the north is permitted to grow southward. Initially, two cases of S-curve channels were proposed for investigation.

The 3B Alternative (Figure 2-5) is based on the present (1998-1999) bathymetry, and the 3C Alternative is based on historical analysis (Chapter 3) where the channel, extending southward, parallels the ocean bar for up to 2.5 miles. Dredging could be minimized for this configuration, as the channel markers could be moved to follow the southward migration. The major drawbacks would be exposure of vessels to turbulent broken waves broadside and requirement by the USCG for the Seattle District to survey the channel prior to relocation of navigation aids. Based on the dangerous wave condition of 3C, only 3B was selected for study. Alternative 3D raised the new SR-105 dike to -2.0 mllw elevation. Alternative 3E consisted of construction of a jetty on the north side of the entrance (Figure 2-5), primarily as a terminal groin or holding structure for sediment, which might enable trapped sediment to move back up the coast with winter storm waves. Alternatives 3F and 3G are similar in orientation to 3A and 3B, with the difference in increased channel width and depth providing increased sediment storage, thus increasing the time between project maintenance operations.

Table 2-2
Definition of Design Alternatives, Willapa Bay Navigation Channel Reliability Study[1]

Alternative	Description	Potential Benefits and Beneficial Uses	Estimated Feasibility
1. Existing Procedure	USCG directed to move channel markers according to natural shift in channel detected in Corps surveys. An S-curved channel could remain.	Least costly O&M alternative	Low – cannot maintain a navigable channel. Would require frequent surveys
2. Modified Existing Procedure	Same as 1, but with dredging performed opportunistically anywhere according to need and to improve (widen) any curved channel condition	May be least costly because of probable low frequency of dredging	Fewer and random beneficial uses. Requires frequent surveys and subject to availability of funds and dredging equipment suited to Willapa Bar
3. Maintain North Channel	Maintain north channel at reasonable cost	Opportunity to place dredged material on North Beach	
3A	Dredge primarily on entrance bar, straight out and fixed in position	Safe navigation route. Possible opportunity for beach nourishment	Most feasible relative to navigation alignment
3B	Modified S-curve (moderate curve)	Possible opportunity for beach nourishment	Feasible if 1,500-ft width available
3C	Modified S-curve (extreme allowable curve)	Possible opportunity for beach nourishment	Not desirable due to difficult navigation
3D	Raise the SR-105 dike (-2 ft mllw) and dredge on entrance.	May reduce erosion on North Beach	Expensive first cost
3E	Construct jetty from tip of Cape Shoalwater and dredge as required	Reduce persistent infilling of channel from the north	Expensive first cost
3F	Same as 3A, except dredge to total depth of 38 ft mllw and with width 1,000 ft	Reduced frequency of maintenance and potential shoaling from the north. Easier navigation	Subject to availability of high-production dredge equipment
3G	Same as 3B, except dredge to total depth of 38 ft mllw and with width 1,000 ft	Reduced frequency of maintenance and potential shoaling from the north. Easier navigation	Relatively expensive due to initial dredging cost
3H-a	Existing SR-105 dike and deepening on potential new channel south of North Channel	If new channel continues to break through, would be beneficial for north shoreline	If new channel breaks through, will be the present channel
3H-b	SR-105 dike raised to -2 ft mllw (3H-b) and deepening on potential new channel south of North Channel	More sheltering for North Cove shoreline; some deflection of flow away from North Channel	Expensive first cost. If new channel breaks through, will be the present channel.
4. Maintain Middle Channel	Maintain Middle Channel at reasonable cost	Shortest route from Bay Center	Enhanced flow to help maintain Bay Center channel
4A	Dredge primarily on entrance bar	Shortest route	If dredged entrance bar, beneficial uses doubtful

(Continued)

[1] All channel depths are 26 ft mllw + 2 ft overdredging (500-ft width at bottom) unless otherwise specified.

Table 2-2 (Concluded)			
Alternative	**Description**	**Benefits and Beneficial Uses**	**Feasibility**
4B	Construct training structure northwest of Bay Center to direct flow away from the North Channel and into the Middle Channel. Dredge on entrance bar only	Create bird islands, oyster grounds; would reduce current along the North Beach	Need to investigate possible changes to navigation at Bay Center and Toke Point; expensive
4C	Construct training structure northwest of Bay Center to direct flow away from the North Channel and into the Middle Channel; dredge primarily on bay (funnel the ebb flow) and dredge on entrance bar.	Create strong Middle Channel; reduce flow in North Channel and erosion on North Beach	Expensive
4D	Construct training structure north of Willapa River entrance to direct flow from North Channel and into Middle Channel. Dredge on entrance bar only	Create strong middle channel; cut off flow in north channel and reduce erosion on North Beach	Need to investigate possible changes to navigation at Bay Center and Toke Point; expensive
4E	Same as 4A, except dredge to total depth of 38 ft mllw and with width 1,000 ft	Reduced frequency of maintenance. Easier navigation	Need to investigate possible impacts on navigation at Bay Center and Toke Point; expensive first cost
5. Maintain South Channel	Maintain South Channel at reasonable cost	--	Doubtful because of lack of data on channel longevity and longer transit for most users
5A	Dredge primarily on entrance bar	Potentially shorter length of dredging to deeper water. Similarity to Grays Harbor	Doubtful because of lack of data on channel longevity and longer transit for most users
5B	Dredge primarily on bay (funnel the ebb flow)	Same as 5A	Considered infeasible because curvature would be too great to capture flow out of the South Bay on ebb
6. Follow Natural Channel	Implement either 3A or 4A according to opportunity	May result in least dredging and safest navigation if careful surveying is done	May be difficult to know when the deepest channel has switched location with stability

Table 2-3
Initial Dredging Volumes Assuming a Static 1998 Bathymetry

Alternative	Short Description	Volume (1,000,000 cu yd)
3A	Straight North Channel	0.8
3B	Curved North Channel, 500- to 2,500-ft width	0.2
3C	Extreme curved North Channel	Somewhat greater than 0.2
3D	Raise SR-105 dike with straight North Channel	0.8
3E	North Channel with jetty	0.8
3F	3A orientation, 38- x 1,000-ft channel dimensions	5.0
3G	3B orientation, 38- x 1,000 ft channel dimensions	4.2
3H	Arm off south side of North Channel at dike	2.1
4A	Middle Channel (bar dredging)	3.0
4B	Bay Center training structure with 4A	2.6
4C	Dredging across whole entrance with 4B	9.9
4D	Willapa River training structure with 4A	2.6
4E	4A orientation with 38- x 1,000-ft channel dimensions	12.0

Note: Unless noted otherwise, channels are 500 ft wide at the bottom and have 1V:3H.

Alternatives 3H-a and 3H-b were selected for evaluation after new bathymetric measurements made during the course of the study indicated some deepening of the southside of the North Channel just seaward of the SR-105 project. The two alternatives were added to determine if any advantages could be found in placing a navigation channel in this vicinity. Alternative 3H-b included raising the elevation of the SR-105 dike from -18 ft mllw to -2 ft mllw, to deflect additional flow through the potential channel location.

Alternative 4 variations were all located at the Middle Channel region (Figure 2-5). The Middle Channel would provide the shortest route for vessel traffic. The alternatives were developed based on evidence that major ebb flow exits the North Channel. Therefore, Alternative 4 variations were designed in an effort to divert flow through the Middle Channel to enhance its persistence at that location. Alternative 4A involved dredging on the bar, which would provide a baseline for the more diversionary alternatives. Alternative 4B provided a training structure just northwest of Bay Center. Alternative 4C added a channel extending seaward from the training structure of 4B to determine if additional dredging might enhance capture of the diverted ebb flow. Alternative 4D consisted of a training dike north of the Willapa River entrance to guide flow directly into the existing bay entrance of the Middle Channel. Alternative 4E was similar to 4A except for the additional sediment storage provided by a deeper and wider channel as was done for Alternatives 3F and 3G.

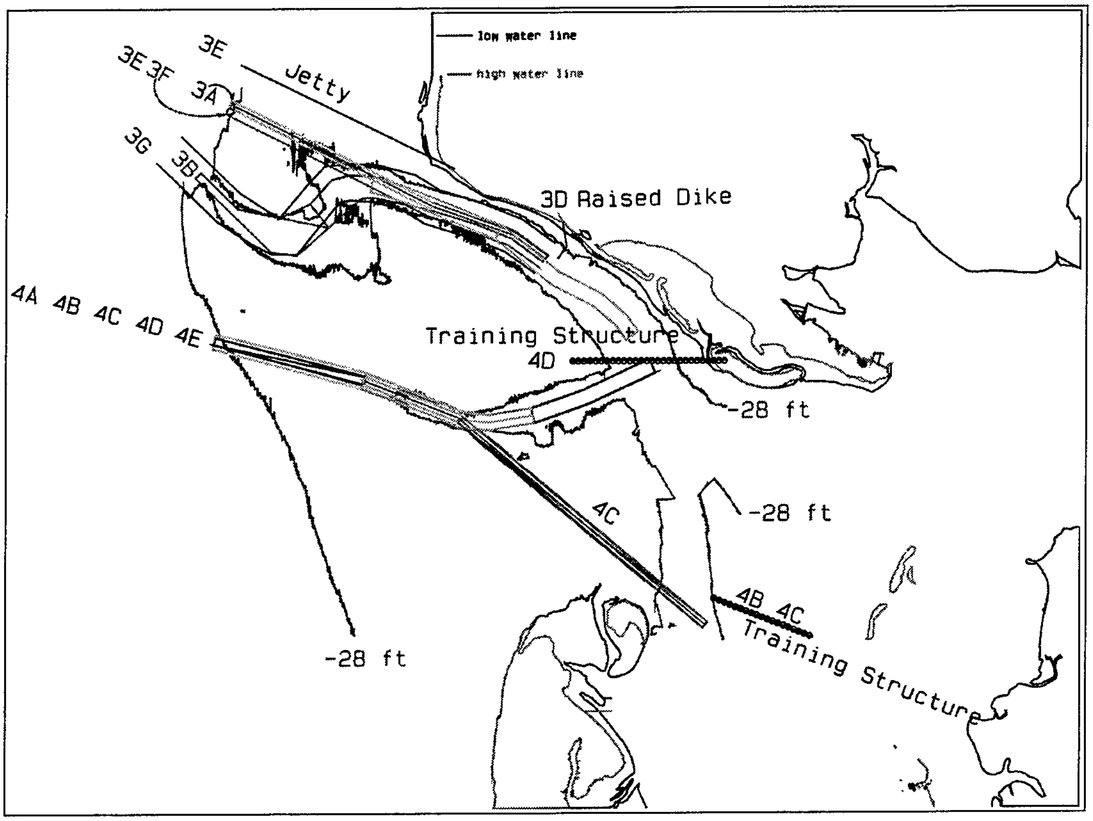

Figure 2-5. Channel and structure location for alternatives

Alternative 5 variations were located at the South Channel. Alternative 5A proposed dredging on the bar side and 5B included bayside dredging. These alternatives were not investigated further because their curvature was considered too great to capture ebb flow out of South Bay and because of the high influx of sediment likely to occur over the entire channel length with winter storm waves.

Alternative 6 would consider implementing Alternative 3A or 4A according to which one would require least dredging. This was not an option to be model studied.

In summary, Alternatives 1, 3A, 3B, 3D, 3E, 3F, 3G, 4A, 4B, 4C, 4D, and 4E were designated for the calculation-intensive tidal hydrodynamics and sediment-transport simulations. A subset of these that showed potential was then examined in the short-wave model. Chapters 5 and 6 describe the short-wave and long-wave simulations, respectively.

Initial Dredging Requirements

A major consideration in the evaluation of channel alternatives by cost is the considerable difference that exists in initial dredging requirements. These differences are examined quantitatively by mathematically cutting design channels into a terrain model based on recent soundings throughout the entrance (the 1998 bathymetry is discussed in Chapter 3).

An estimate of maximum likely uncertainty in the calculated cut volumes is 100,000 cu yd/mile (equivalent to about a 2-ft uncertainty in the 500-ft-wide design). In addition, actual volumes that will be required to create the selected alternatives may differ from those calculated here because the bay entrance is continually being remolded. Errors arising from imprecise knowledge of the 1998 bathymetry are probably small compared to changes that could occur prior to actual dredging. The temporal changes do not, however, overshadow the wide range of cut volume requirements among design alternatives. Calculated volumes are rounded to the nearest 100,000 cu yd in Table 2-3 and still reveal major differences between the alternatives. The initial dredging calculations thus represent reasonable criteria for evaluating initial costs.

Certain calculated differences (e.g., between straight and curved alternatives in the North Channel) also indicate the magnitude of changes in dredging requirements that could occur over a few years. Differences among the calculated values also indicate the magnitude of change in dredging requirements depending on whether the new channel is maintained in a fixed location or allowed to shift with the natural channel (natural migration patterns are discussed in Chapter 3). Following the natural migration would be feasible only within prescribed bounds. When the natural channel approaches unsafe conditions, it would be advisable to establish a completely new outlet farther north where nature began erosion of new outlets in 1941. The cost of maintaining such a moving channel depends on how well natural migration cycles can be anticipated, how soon unsafe conditions recur, and how much extra dredging would be required to force a new outlet ahead of nature's schedule. That extra effort might be similar to the increase in cut volumes going from Alternatives 3G to 3F. Historical channel migration patterns are analyzed in Chapter 3.

Channel designs were based on two cross-sectional dimensions. One provides 26-ft depth plus 2-ft over dredging across a 500-ft width. The other provides 38-ft depths over a minimum 1,000-ft width. Alternative 3B provides 500-ft widths along the straight reaches, but a 1,500-ft minimum width through the curved reaches. An informative illustration of 1,000-ft versus 500-ft widths in Figure 2-5 can be seen along the outer reaches of Alternatives 3A versus 3F and the outer reach of Alternatives 4A and 4E. All designs include 1V-on-3H slopes between the base of the channel and the 1998 sediment surface. Table 2-3 gives the volume calculations for alternatives shown in Figure 2-5. Navigability and alteration of hydrodynamics by dredging were considered in placing these design channels. Minimizing initial dredging was a secondary consideration.

The initial dredging requirements for the 28-ft channel through the middle of the entrance (Alternative 4A) could have been reduced by reorienting the design to the shortest path over the bar rather than to the navigationally preferred alignment shown in Figure 2-5. This less desirable southwest alignment would have reduced the dredging by only about 13 percent and thus is not shown in Figure 2-5, or Table 2-3, or considered further in this report.

For the curved North Channel alternatives (3B and 3G), safe passage of barges on a tow requires a minimum width of 1,500 ft on the curved reaches. Because the naturally curved channel exceeded 28-ft depths over a width that was greater than 1,500 ft in 1998, volume changes were also calculated with a design width that ranged between 500 and 2,500 ft as shown in Figure 2-5. The total cut volume for Alternative 3B was only 200,000 cu yd. By this approach,

Alternative 3B required less dredging than any other alternative and gave considerable extra width in the curves.

For safe passage by barge traffic, the design width of Alternative 3G (the curved 38-ft alternative) was also expanded beyond the 1,000- to 1,500-ft minimum for the straight and curved reaches. A 2,500-ft width was adopted along the entire ocean extension of 3G. The resulting cut volume was only slightly larger (8 percent) than the volume obtained for the 1,000-ft-wide straight alternative (3F).

In summary, Table 2-2 describes the design dimensions for each identified alternative. In calculating initial dredging requirements, these widths were implemented except in the case of the two curved alternatives (3G and 3B). A uniform 2,500-ft width was assigned for Alternative 3G, and a variable width (500 to 2,500 ft) was assigned in calculating the smallest of all the cut volumes (Alternative 3B). These widths illustrate how much wider the 28- and 38-ft clearances could be with little additional dredging beyond that needed for the minimum safe width. The resulting small inflation of calculated volumes also better represents the amount of initial dredging that might be incurred when conditions are not so favorable as obtained with the chosen design channel placements on the 1998 bathymetry. Further such analyses could be performed as new alternatives are identified and existing alternatives are defined.

Navigation Channel Alternatives and Disposal Sites

The location and method of disposal are central factors for evaluating the economic feasibility of potential dredged-material disposal sites. The dredged material is expected to consist primarily of sand. Disposal sites were evaluated relative to proximity to environmental permitting and coordination requirements, environmentally sensitive areas, biological resources, and resource agency requirements. Appendix H describes previously and currently permitted disposal sites, potential new rehandling disposal sites for beneficial reuse, cost of disposal, possible environmental concerns, site capacity, and disposal equipment requirements.

The study determined that more than one disposal site is feasible for the various dredging alternatives. A preliminary attempt was made to assign the disposal site alternatives to the navigation channel dredging alternatives. The matching of channel dredging and disposal site alternatives was a logistical and iterative process that accounted for cost of disposal, schedule of construction, site capacity and volume of dredging, and dredging and disposal equipment compatibility. These preliminary dredging site and disposal site combinations are presented in Table 2-4. Figure 2-6 shows approximate location.

The identified disposal sites, as well as the combination of disposal sites and navigation channel alternatives, are preliminary and would be specified during permitting and design. These dredging site and disposal site combinations may be modified upon possible additional numerical modeling and if new information about channel sedimentation and volume of future maintenance dredging becomes available.

Table 2-4
Dredging Site/Disposal Site Combination Alternatives

Channel Alternative	Dredging Volume, cu yd	Description	Disposal Site[1]	Estimated Dredging and Disposal Cost[2,3] per cu yd
3A	800,000	Dredge primarily on entrance bar, straight out and fixed in position.	A – Beach Nourishment B – Beach Nourishment D – Nearshore Berm E – North Channel Disposal	$8.70 $8.70 $6.60 $4.30
3B	200,000	Modified S-curve (moderate curve).	A – Beach Nourishment B – Beach Nourishment D – Nearshore Berm E – North Channel Disposal	$8.70 $8.70 $6.60 $4.30
3H-a, 3H-b	1,600,000	Added after 1999 survey to coincide with the straight path seaward from near SR-105 project that had minimal change since 1998 survey. Two variations include existing SR-105 (3H-a) and SR-105 dike raised to -2 ft (3H-b).	A – Beach Nourishment B – Beach Nourishment D – Nearshore Berm E – North Channel Disposal	$8.70 $8.70 $6.60 $4.30
4A	2,300,000	Dredge primarily on entrance bar.	F – Goose Point I – Side Casting	$6.80 $3.80
4E	12,600,00 0	Same as 4A, except dredge to total depth of 38 ft and with width 1,000 ft.	F – Goose Point I – Side Casting	$5.90 $3.50

[1] Figure 2-6 and Appendix H give locations of disposal sites.

[2] Each cost estimate is based on disposal of amount shows in second column, although in some cases, it may be advantageous to dispose of different portions of this amount at more than one site in a given year.

[3] Cost estimate does not include mobilization and demobilization.

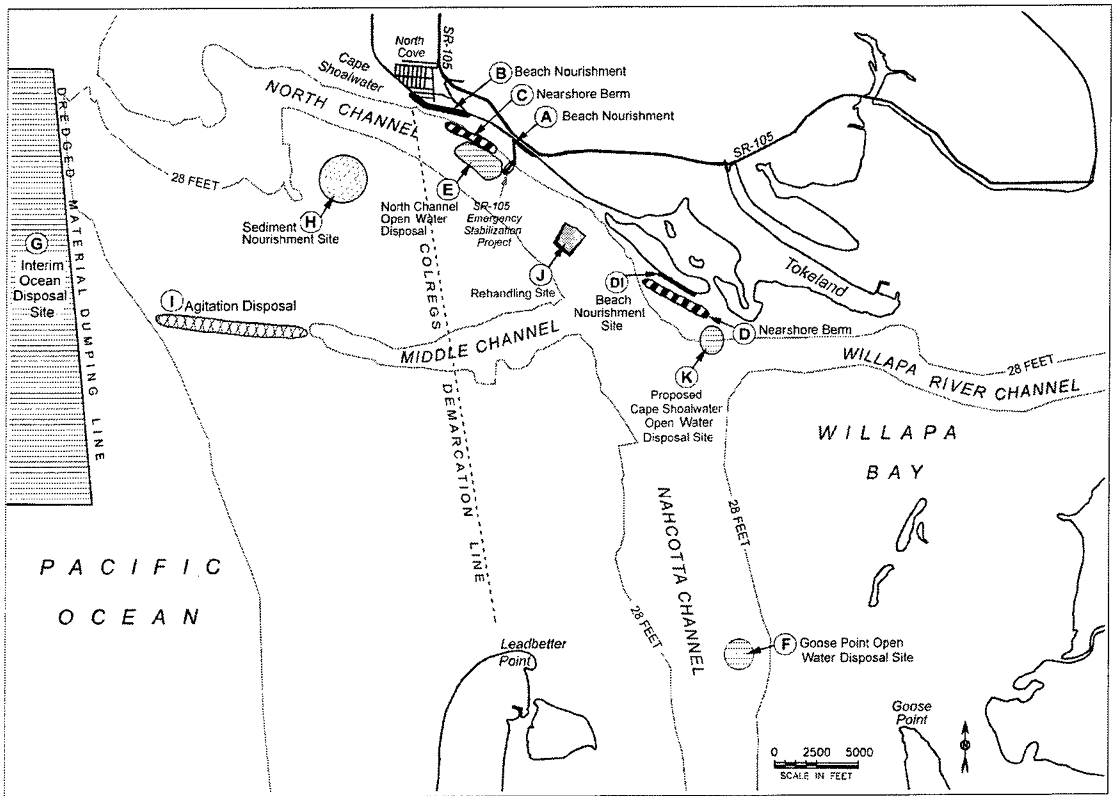

Figure 2-6. Potential dredged-material disposal sites and means of disposal

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3 Geomorphology[1]

Introduction

Geomorphology is a science that systematically examines, classifies, and interprets landforms. This chapter presents results from a geomorphic investigation of the entrance to Willapa Bay that included recent and historical bathymetric changes. It also investigates the implications of large-scale, relatively longer term changes (on the order of 1 to 100 years) on channel feasibility and design. The basic sources of information are charts and aerial photographs, which are numerous and cover more than a century of dynamic change.

The study area extends eastward from the outer edge of the ebb shoal to the inner bay region where channels are oriented north-south, roughly at the meridian of Toke Point (Figure 2-1). Cape Shoalwater and Leadbetter Point bound the entrance on the north and south, respectively. Channels and shoals in the entrance are subject to rapid changes induced by large waves and strong currents. Larger scale changes nevertheless include trends and cycles that are well documented by the charts and photographs discussed in this chapter.

Setting

The coast of Washington extends from Cape Disappointment at the entrance of the Columbia River northward to Cape Flattery at the entrance of the Straits to Juan de Fuca (Figure 3-1). The southern half of this 150-mile-long reach follows a relatively linear north-south line reflecting the marine-fluvial origin of sediments that have smoothed the coastline between two rock outcrops, Cape Disappointment north of the Columbia River and Point Grenville at the opposite end. The broadly north-south shoreline can be segmented into three slightly concave westward segments by ebb shoals at Grays Harbor, Willapa Bay, and the Columbia River.

This reach of coast lies south of the extent of Pleistocene glaciation. The 60-ft-depth contour runs north-south about 2.7 miles from shore giving a nearshore slope of 0.004 except where the contours bulge seaward around deltaic deposits off the Columbia River, Willapa Bay, and Grays Harbor. The heads of submarine canyons farther offshore (Astoria, Willapa, Guide, and Grays) are at

[1] Written by Mr. Edward B. Hands, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS.

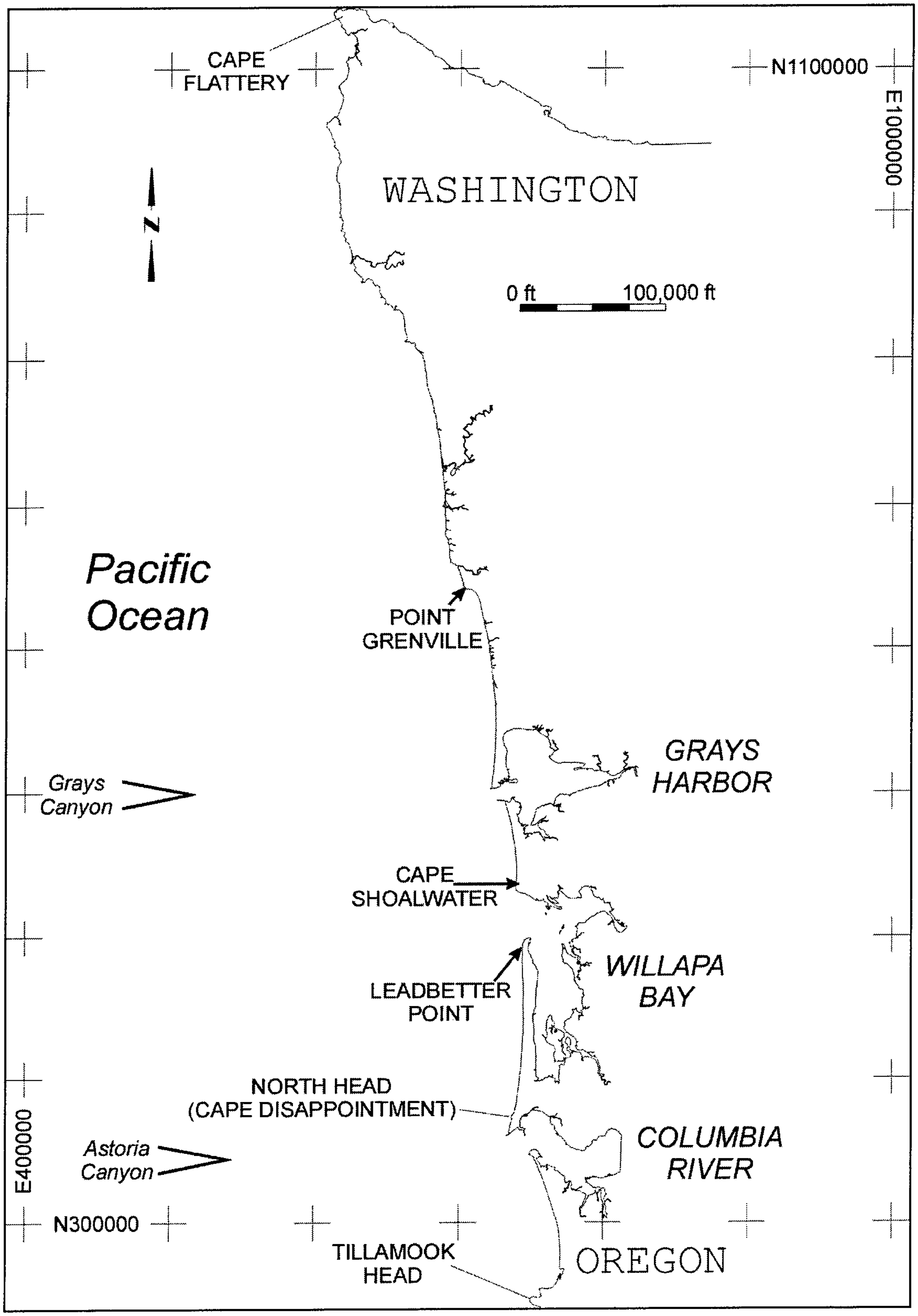


Figure 3-1. Regional location map

depths of about 600 ft. These canyons are too deep to interact significantly with waves or sediment transport on the scale of interest here. Beach sand samples taken between the Columbia River and Grays Harbor range from about 0.15- to 0.25-mm in median diameter.[1] Samples taken from the Willapa Bay entrance bar by the U.S. Army Engineer District, Seattle, in the 1960s, 1970s and for this study reveal consistent textural characteristics: well-sorted, fine-grained, angular sand (0.18-mm median diameter and 0.3-0.5 sorting).

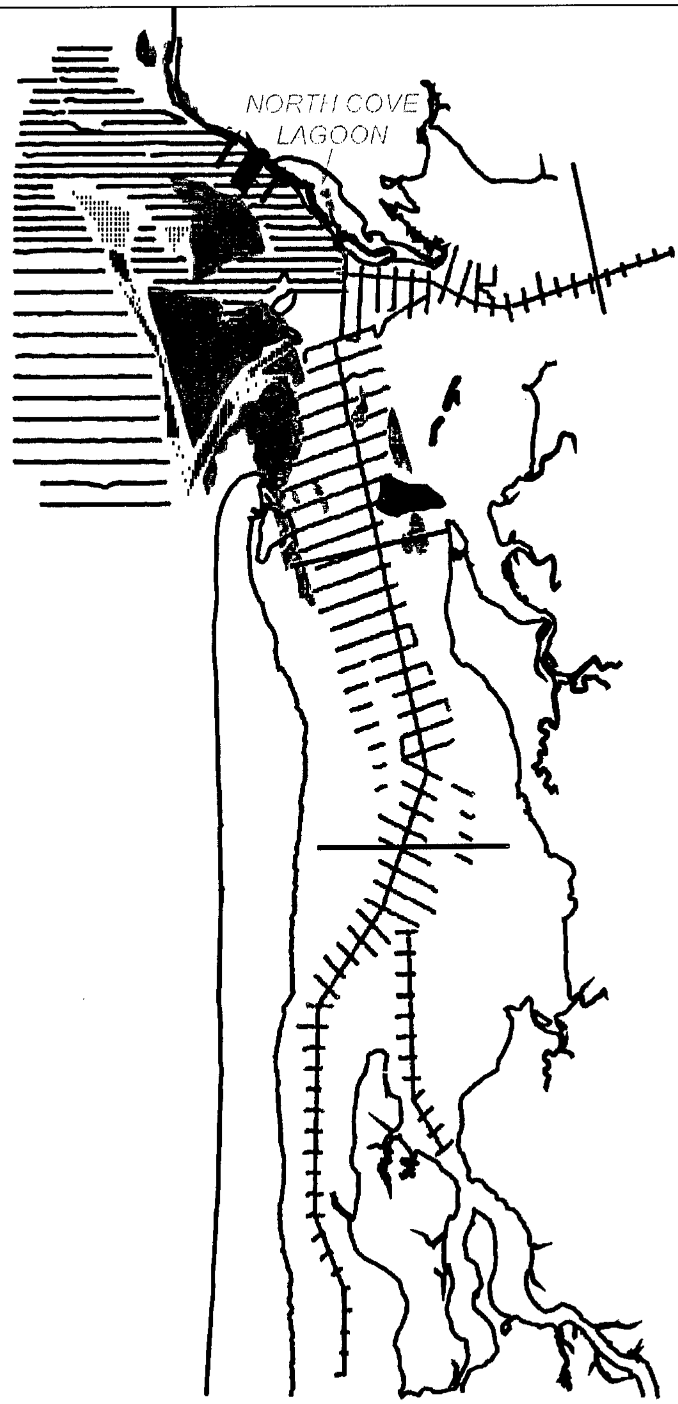
Bathymetric Data Used in Analysis

The Seattle District has conducted hundreds of hydrographic surveys of portions of Willapa Bay. Almost annually between 1927 and 1978, the Seattle District prepared charts covering the entire entrance with soundings taken each year (primarily in July and August). At less frequent and irregular intervals similar charts were prepared between 1852 and 1922. After 1967, when Federal maintenance dredging for the Willapa River channel ceased, Seattle District surveys were curtailed to the portion of the entrance near the navigation channel. Though regular dredging at the entrance ceased in 1974, a small (80,000 cu yd) experimental channel was dredged in the middle of the entrance in 1997. Fifty-six full-entrance charts, compiled between 1852 and 1978, were obtained from District archives to support this study together with restricted-area channel surveys for the years 1981, 1982, 1983, and 1984 and District digital hydrographic data taken between 1993 and 1997. No surveys were obtained for the period between 1985 and 1992. Less detailed information from this period was obtained from aerial photographs.

As part of the field data collection effort for the present study (Chapter 4), the most dense and extensive hydrographic survey to date was conducted of the whole bay by combining synoptic soundings taken by Seattle District medium- and shallow-draft survey boats, contractor vessels, and the Scanning Hydrographic Operational Airborne Lidar Survey (SHOALS) of the U.S. Army Corps of Engineers (Lillycrop, Irish, and Parson 1997). With the SHOALS system, elevations were surveyed over extensive shallow to slightly emergent shoals that cover much of the central portion of the bay entrance. Being the site of almost continual wave breaking even in the summer, the outer portions of these shoals had never before been surveyed. Even landward of the breakers, precise elevations over broad areas were unknown because the areas were too shallow for survey boats.

Because of the turbidity in the bay, the extent of SHOALS coverage was limited primarily to depths less than about 10 ft. Elevations were obtained, however, in previously unsurveyed areas, which significantly improves the basis for sediment volume and cross-sectional area calculations for analysis of sediment volumes and tidal discharge. By combining data from various surveys as described, nearly 500,000 accurate survey points were obtained over the 40,400 acres of the bay entrance. Another 100,000 less dense points were also sounded in 1998 to determine bathymetry in the lower bay and the lower Willapa River (Figure 3-2).

[1] Personal Communication, July 1998, Mr. M. Buijsman, Washington Department of Ecology.



Symbol	Soundings	Platform	Dates
	299,749	SHOALS	21, 26, 27 Jul 1998
	130,402	Shoalhunter	6 - 20 Aug 1998
	102,925	Shoalhunter	29 Jul - 1 Sep 1998
	37,463	CE Small Boat	17, 22, 29 Sep1998; 16 Oct 1998
	28,597	PIE Boat	21, 26, 27 Jul 1998
	451	Cpt. Landreth	21, 26, 27 Jul 1998

Figure 3-2. Soundings taken through Willapa Bay from July to October 1998

Based on evaluations at other sites, the accuracy of the SHOALS data is reported to be ±3 m horizontally and ±0.15 m vertically.[1] Boat survey accuracy is expected to be ±1 m horizontally and ±0.67 m vertically.[2]

As part of the data collection for this study, the bottom was surveyed again in the spring of 1999. The 1999 survey covered only the entrance area where channels are proposed and was conducted by the Seattle District survey boat *Shoalhunter*. In the interval between these two major surveys, Pacific International Engineers[PLLC] and Evans-Hamilton, Inc., took additional soundings in selected areas near instrument stations, in areas of active scour and deposition (near the State Route (SR) 105 dike, in the borrow area), and in more rapidly changing areas along the north channel.

The 1998 and 1999 surveys and the sequence of historical bathymetry covering (at uneven intervals) 147 years reveal processes, trends, and cycles that provide information on long-term, wide-area changes in sediment and fluid dynamics. All full-entrance charts consulted in this study are reproduced at reduced scale in Appendix A except for those years for which there are multiple charts, in which case only the most informative chart for the year is shown.

Comparison of Channel Alternatives by Fairways

Location of historical channels relative to present alternatives

The alternatives considered in the search for a more reliable channel can be divided into three subsets with every Alternative Channel (Table 2-2) near one of three locations: the North Fairway, Middle Fairway, and South Fairway as depicted in Figure 1-3. Channels tend to occur in or near these fairways historically. Each channel alternative has a specific design cross section; the fairways are much broader zones and can contain several channel alternatives. Many features of the proposed channels will be evaluated in terms of geomorphic conditions that can be documented for these three fairways. The conceptual fairways and Alternative Channels are denoted by mixed-case lettering to distinguish them from evolving natural channels in the north, middle, or south parts of the bay denoted by lowercase lettering.

Channel alternatives will be compared in terms of relative values of several channel parameters that have been tabulated from the bathymetric record. The frequency of occurrence of natural channel parameters in favorable ranges for each fairway can be compared with those of the Alternative Channel design parameters to obtain a relative ranking of the engineering effort that would be required to establish and maintain each proposed Alternative Channel.

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[1] Personal Communication, September 1998, J. Lillycrop, SHOALS Center of Expertise, Mobile, Alabama, also see Guenther, Thomas, and LaRocque (1996).

[2] Personal Communication, December 1998, H. Arden, U.S. Army Engineer District, Seattle, Seattle, WA. These error estimates refer to uncertainties in individual points. Biases have not been investigated.

Tabulated channel parameters

The parameters tabulated for this comparative analysis of alternatives are listed in Table 3-1. The datum for elevations is mean lower low water (mllw). Except for the historical charts, which were created with Corps-surveyed shorelines, all figures show the high-water line (hwl) and the low-water line (lwl) determined from 1996 aerial photography and U.S. Geologic Survey (USGS) quadrangle sheets. Figure 3-3 shows a typical location where the geomorphic parameters were evaluated in a particular representative year for three *reaches* along the North Channel. Reach 1 always refers to channel outlets over an ocean bar. Four such *outlets* are shown under Reach 1 in Figure 3-3. Additional reaches are defined to accommodate major changes in channel orientation or size moving inland from the bar to Reach 2 and then to Reach 3 farther upstream. Similar measurements were made for each channel and identified by date, fairway, and reach for each survey and for each of one to four outlets if the channel split before reaching the ocean as the north channel does in Figure 3-3. These parameters were also tabulated from the individual channel charts of the 1980s (Appendix A). To more appropriately contrast the proposed Alternative Channels, however, the database for the following comparative analysis is restricted to data tabulated from the full-entrance surveys that charted all clear channels through the entrance between 1852 and 1978. Geomorphic parameters showing the strongest contrasts among channel alternatives are discussed next.

Table 3-1
Geomorphic Parameters Measured in Each Outlet and Each Channel Reach of Each Fairway on Each Chart of the Willapa Bay Entrance

Measurement	Description
Depth, ft	Limiting depth over the bar and representative depth for upstream reaches
Orientation, deg	With respect to grid north
Width, ft	Between 24 ft mllw depth contours
Coordinates, lat, long	Of the bar or of the centroid of the upstream reaches
Longevity	A count of the number of times a particular channel, reach, or outlet appeared on one of the annual summary charts regardless of channel depth or width.

Differences in channel longevity

A *clear* channel is taken here to be one charted from the ocean bar eastward entirely across the entrance shoals into the back bay regardless of specific channel depth or width. Depth and other channel parameters are examined after first considering channel longevity. By this approach, the number of historical categories is initially restricted to three fairways, and the results are not biased toward any one of the channel alternatives (e.g., one with a large versus small cross section). After considering longevity, the next sections examine questions of how often and closely the historical channels matched the proposed design alternatives.

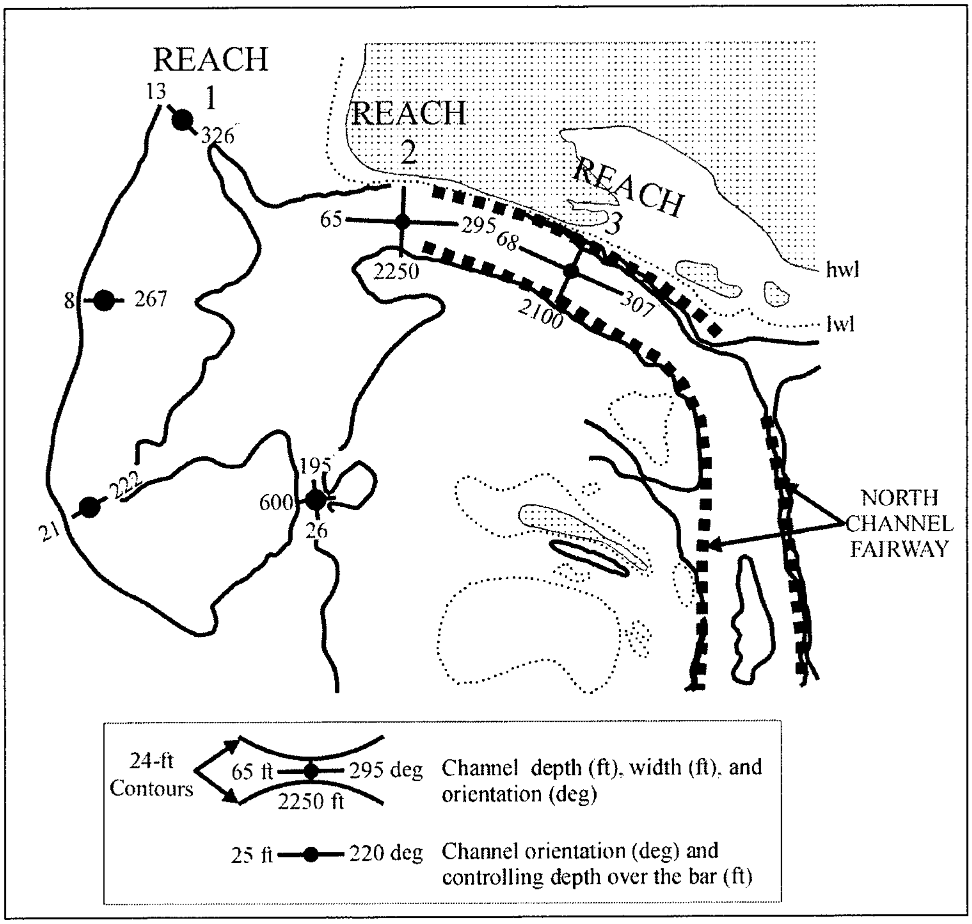


Figure 3-3. Definitions of channel reach and tabulated geomorphic parameters

If the Seattle District identified a channel worthy of charting across the bay mouth, it is considered to have been clear at that time. If no channel was charted in a particular region, it is assumed none existed. This assumption is considered reasonable because the charts selected for the comparative analysis portray conditions over the entire entrance. Journals of some early explorers (quoted in Swan 1857) and a chart of shipwrecks in the bay entrance (Kranz 1986), suggest that sailing vessels often attempted to enter Willapa Bay through the southern half of the bay. Later in the 1980s and early 1990s, when surveys were restricted to navigationally important reaches, channels in the middle or south fairway may have escaped inclusion on charts and be underrepresented in the longer period channel database. Therefore, comparison of channel parameters among the fairways will be restricted to the 1852-1978 period during which surveys covered the full entrance area.

Even when the full north-south extent of the entrance was surveyed, wide areas within it were not sounded because they were too shallow for the survey boat or were surrounded by breakers. The absence of elevations for these areas would hinder calculation of precise entrance cross-sectional areas and sediment

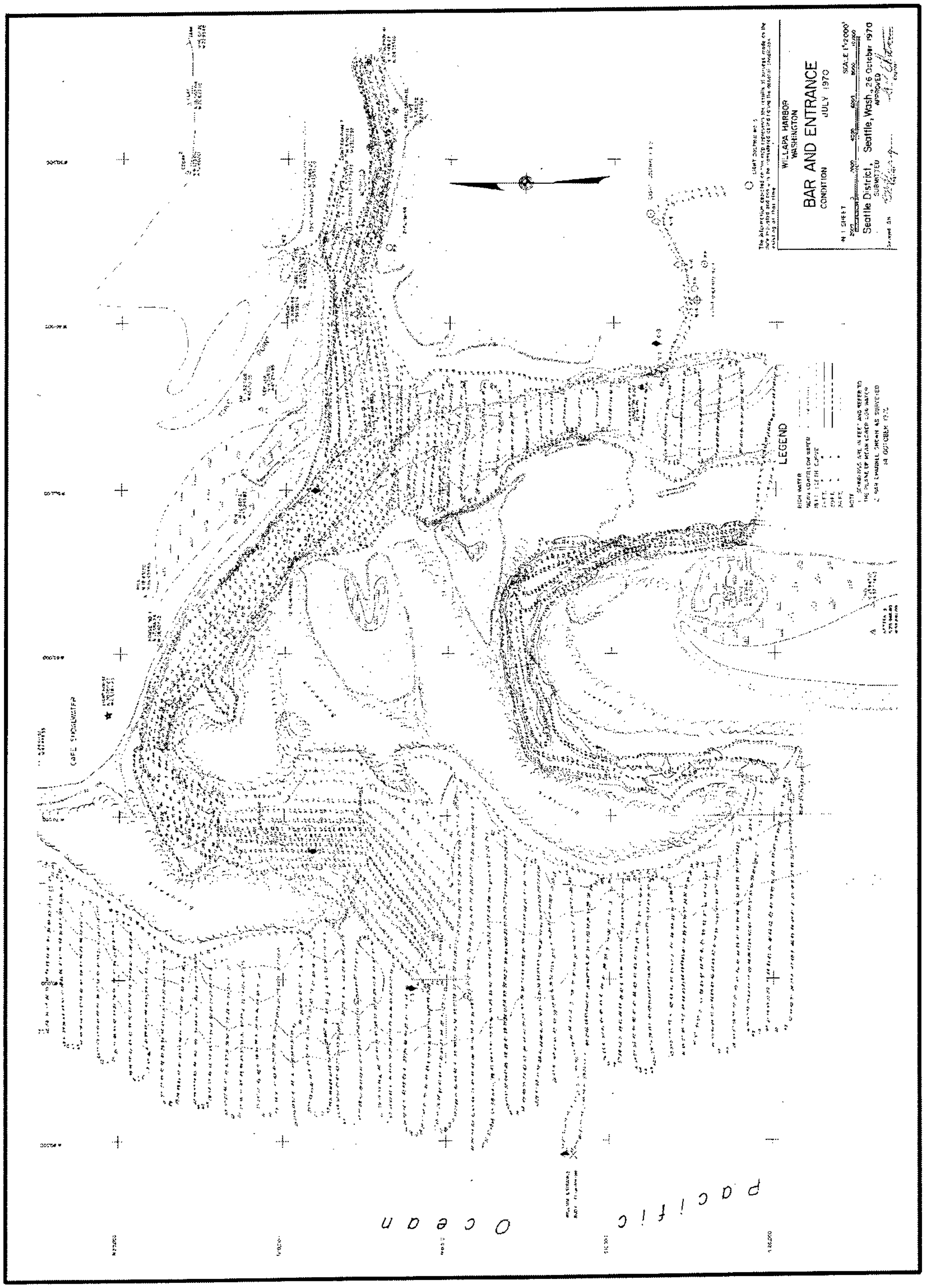


Figure 3-4. One of the 64 historical charts forming the basis for the geomorphic evaluation

volumes for those periods, but does not mask any navigationally significant channel. Figure 3-4 is an example of the historical charts of the full entrance area. One such chart for each year being considered appears at a reduced scale in Appendix A along with restricted area surveys of the 1980s and 90s and the detailed surveys taken to support this study.

Considering the more restrictive database (the 56 full-entrance charts, 1857 to 1978, selected for a balanced fairway comparison), the Middle and South Fairways had clear channels on less than 20 percent of the charts (Figure 3-5), whereas the North Fairway was not only always clear, but was larger than any of the channels in the other fairways (Figure 3-6). Therefore, it is concluded that any navigable channel in the Middle or South Fairways would require substantially greater long-term maintenance than one in the North Fairway unless the natural tidal patterns were significantly altered.

The extra effort of maintaining a channel in the Middle or South Fairway may be justified if these fairways offer superior benefits. Geomorphic criteria that might suggest such benefits are examined in the next sections.

Differences in channel depths and orientations

As can be seen from Figure 3-6, the North Fairway has depths closest to and sometimes exceeding the design depth of 28 ft mllw (26 ft plus 2 ft of advance dredging and overdredging). As discussed in Chapter 2, the orientation of vessels with respect to the waves is a critical risk factor for crossing the bar. Based on wave climatology to be presented in Chapter 5, the 45-deg sector centered on 270 deg is considered most desirable for all three fairways. Figure 3-7 shows that this condition has occurred most frequently in the North Fairway with the South Fairway coming in second ahead of the Middle Fairway, which was often too far counterclockwise or south of the favored westward orientation.

Figures 3-8 and 3-9 show the temporal changes in depth and orientation addressing each fairway separately. Weak interactions between depth and orientation are evident. Persistence, trends, and cycles in past values can be examined in these figures and will be interpreted in a later section as indicative of likely future conditions.

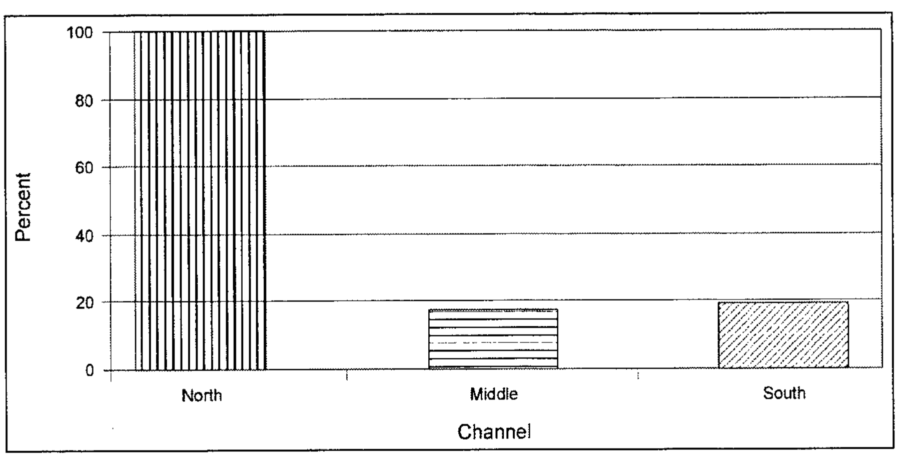


Figure 3-5. Frequency of fully-opened channels

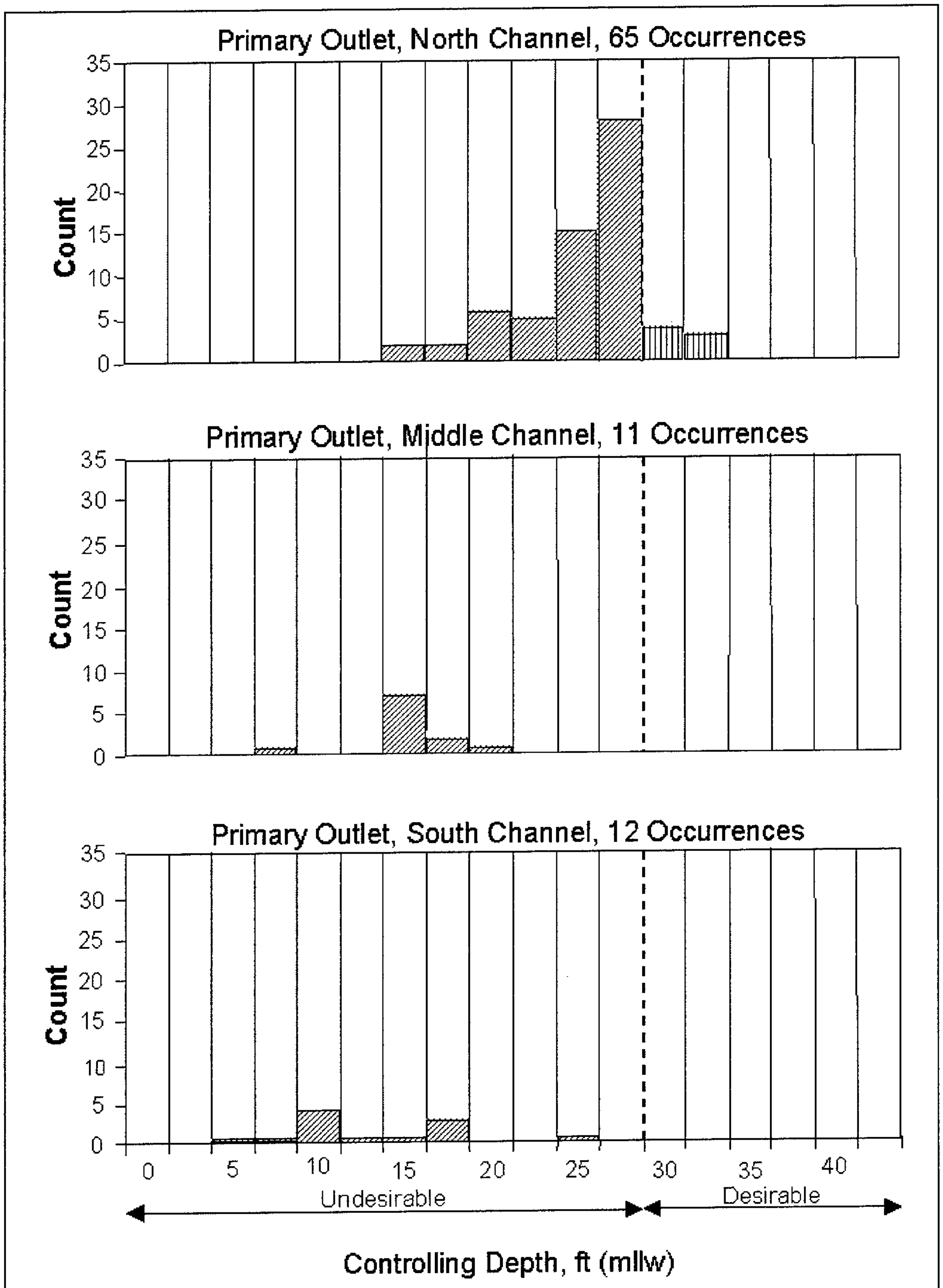


Figure 3-6. Channel depths by fairways

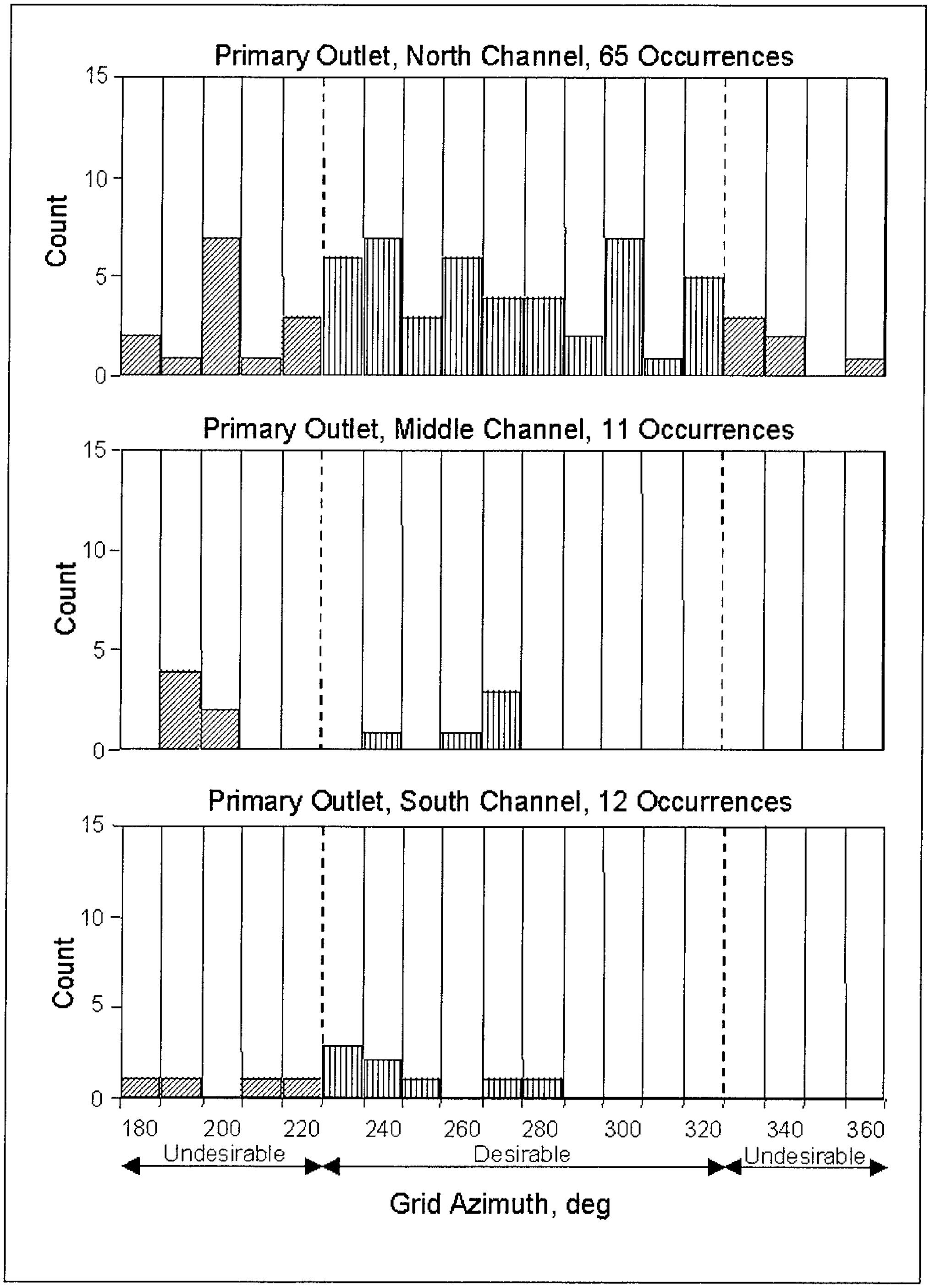


Figure 3-7. Channel orientation by fairways

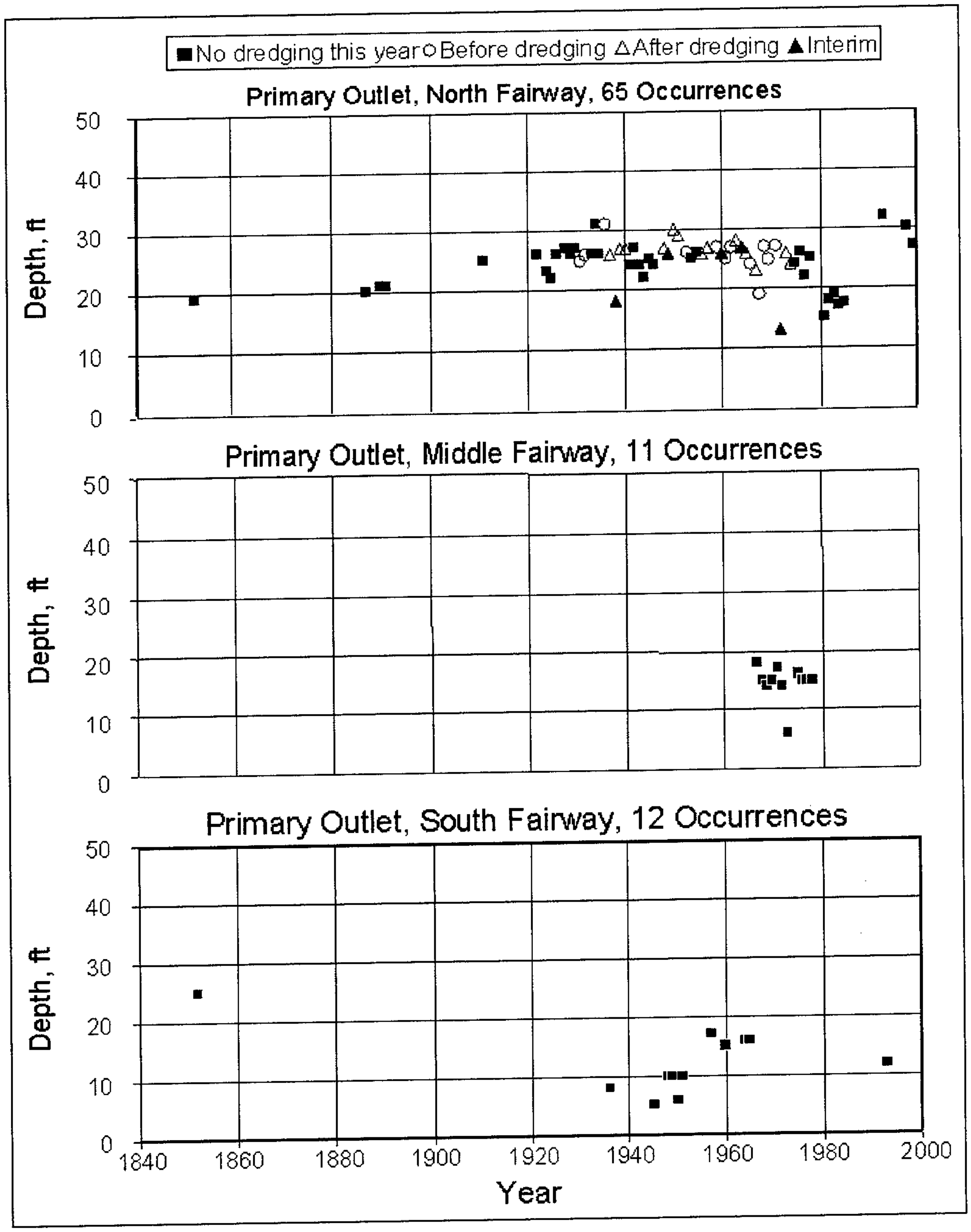


Figure 3-8. Temporal variations in bar depth

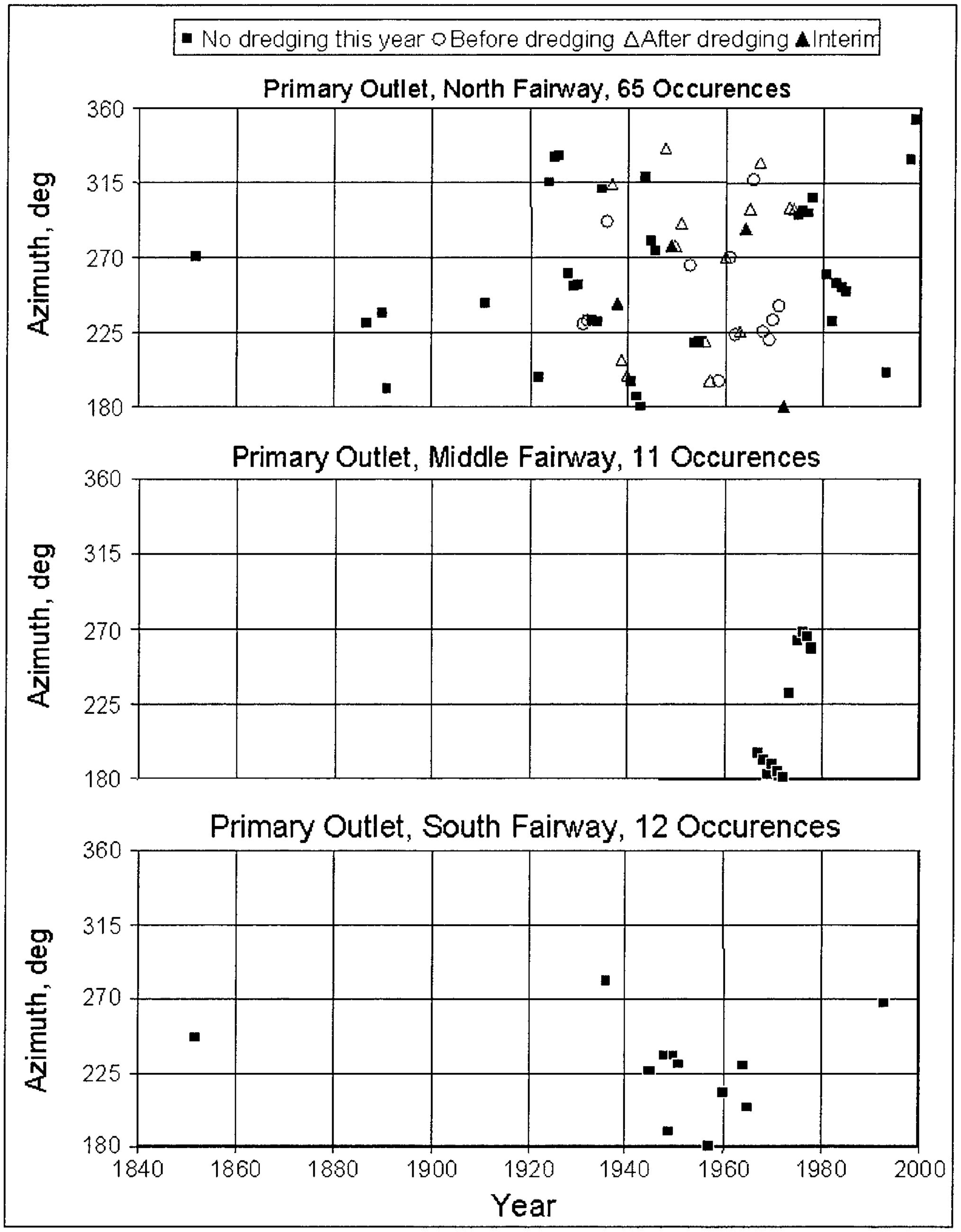


Figure 3-9. Temporal changes in bar channel orientation

Though depths were significantly different among the fairways (Figure 3-6), depths within fairways were relatively stable in time (Figure 3-8). The north channel bar was dredged from 1931 to 1941, again from 1949 to 1974. As presented in Chapter 2, the dredging volumes were not large, typically less than 350,000 cu yd. The bar depths shown in Figures 3-7 and 3-8 for these years might be deeper because of dredging. One should note in the time series (Figure 3-8), however, that the primary bar in the North Fairway was at times deeper than 25 ft before, between, and after these two periods of dredging. Also note that depth symbols are coded to indicate if that year's chart was prepared before or after dredging; there is no significant increase in depth that can be related to dredging. So, the dredging contribution to charted channel depth is small and cannot invalidate the observation that the North Fairway has depths naturally deeper than the other fairways.

At different times, the North Channel split into multiple outlets (Figure 3-3). Some years, there was only one outlet, whereas during other years, there were as many as four. The primary bar is the outlet providing the greatest controlling depth. Only the *primary bar* is shown in Figure 3-6. Middle and bottom panels of Figure 3-6 show the same parameters evaluated similarly for the other two fairways. Again, note the relative stability of depth with time.

Only a single instance of simultaneous channels in different fairways occurred prior to 1928, but multiple fairway channels became common after the 1930s. As can be seen in Figure 3-8, the entrance was charted less frequently prior to 1926 than afterwards. The lack of any Middle and South Channels during this earlier period suggests that Middle and South Channels were at least less common during the pre-1928, poorer-sampled period. The entrance to Willapa Bay has persistently widened from about 4.5 miles (7.2 km) in 1926 to 6.5 miles (10.5 km) along the same transect in 1980. Possibly, the Middle and South Channels were less common earlier because of the narrowness of the entrance. The relationship between the entrance width and number of channels suggests, however, that the change from single to multiple channels occurred abruptly around 1928, rather than multiple channels gradually becoming more common and as the width increased.

Geomorphic Cycles

Cycle identification

A compilation of approximately 1:60,000-scale charts submitted to the District Engineer in 1949 (U.S. Army Engineer District, Seattle 1949) indicates long-standing awareness that the Willapa entrance channel migrated from year to year for long periods. This observation did not appear in the published literature prior to 1965. Andrews (1965), Terich and Levenseller (1986), and Shepsis, Hosey, and Phillips (1996) published accounts of the channel movements relating them to shoal growth that deflected the main channel southward, but documentation and analysis of the number and duration of channel cycles were still lacking. Recent inspection of the longer bathymetry record allows full documentation (at approximately yearly intervals) of seven cycles of spit elongation and channel migration between 1933 and 1998. Three additional cycles will also be identified from earlier charts. The earlier events are, however,

less fully recorded because, prior to 1920, surveys were conducted much less frequently. Additional cycles between 1887 and 1920 may have been missed between the infrequent early surveys.

Along more than a mile of bay shore, where the North Channel skirts past Cape Shoalwater before entering the Pacific Ocean, waves and currents have eroded the shoreline for at least the recorded 147 years at rates in excess of 100 ft/year (Richey et al. 1966, Komar 1998, Terich and Levenseller 1986). Between 1852 and about 1940, this bay shore recession completely eroded two peninsulas (with three former lighthouses). From the charts, these peninsulas appear to have been low-elevation sand spits that were built back into the entrance by southbound longshore transport entering the bay from ocean beaches to the north. Around the turn of the century, northward migration of the main channel began to rapidly erode these spits. Inspection of the historical charts indicates that the long-term rate of shore recession increases westward from the North Cove lagoon to the ocean shore (Appendix A).

The outer receding bay shore has thus rotated clockwise around a pivot point near the cove. Lowell (1997) reports that receding outer bay shoreline is encountering increasingly older, higher, and more consolidated sediments. The more resistant sediments may be similar to those present in the adjacent, relatively stable area behind the North Cove lagoon. Throughout most of this century, however, the bay shore recessed rapidly at a fairly constant rate. Only recently does this rate appear to decline.

In contrast to northward recession of the bay shore, the outer portion of the North Channel typically migrates southward across the ebb shoal, deflected by the shore-tied submerged spit growing from Cape Shoalwater. Periodically, channel migration is interrupted by spit dissection, which allows ebb currents to flow directly seaward out of the North Channel. Dissection always starts with erosion of a notch on the landward side of the submerged spit. At such times, multiple incipient outlets often form. Typically, the notch or notches will widen and extend oceanward for several years until depth across the entire spit reaches 18 ft, at which point the distal end of the spit (which is now an isolated shoal) begins to migrate to the southeast. The shoal migration rate is relatively rapid compared to extension of the spit. The new outlet captures the majority of the North Channel discharge and the other outlets gradually fill. The shoal eventually merges with others in the middle portion of the bay entrance.

Figure 3-10 illustrates the second half of a cycle that began several years earlier when the dominant discharge flowed directly seaward through a channel that over the years rotated clockwise to the north-south position shown in the top panel. Note that a vessel could avoid depths less than 18 ft by following this north-south channel except for a short passage over a saddle near the center of the entrance. In the next phase in 1941, top panel, several new outlets appeared. By 1945 (middle panel), the distal end of the spit had been isolated by a breach entirely across the spit. By 1949 (lowest panel), the spit-derived shoal had migrated southwest and merged with inner entrance shoals. All the while, the submerged ebb-edge spit continued building southward from Cape Shoalwater.

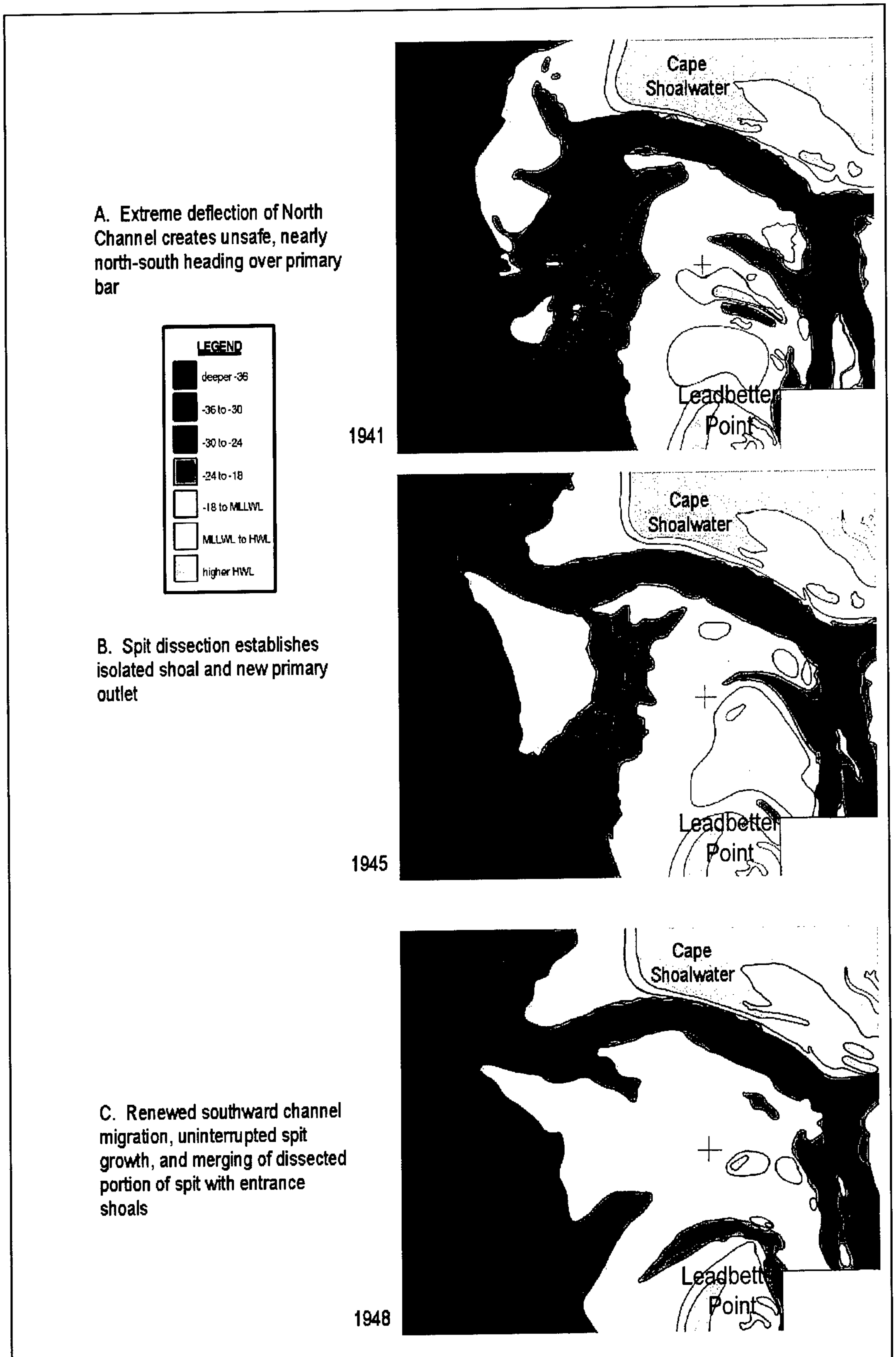


Figure 3-10. Second half of one channel cycle

Figure 3-11 shows a 20-year series of changes that includes two cycles of spit elongation and channel deflection. Cycles have been continuous throughout the 147-year period that the entrance has been charted. Eleven such cycles can be documented between 1887 and 1997.

Repeated cycles of spit growth influence not only the position of the bar channel, but its depth and alignment, the size and location of entrance shoals, and the erosion rates along the North Cove shoreline. Figure 3-12 shows changes in the rate of shoreline retreat along the North Cove relative to the cutting of two new north channel outlets.

Climate and geomorphic controls

Peruvian fishermen use the term El Niño to refer to warming of their ocean fishing grounds around December during certain years. Climatologists independently coined the term Southern Oscillation to refer to an atmospheric condition in the Southern Hemisphere that explained occasional failures of the Indian monsoon. The term El Niño-Southern Oscillation (ENSO) refers to the more recent recognition that both phenomena are expressions of the same coupled ocean-atmosphere, global phenomenon. Prediction and identification of the full impacts of the ENSO are the subject of intense research.

Spit dissections at Willapa Bay follow ENSOs, but dissection occurs much less frequently than ENSOs. When spit elongation deflected the channel and possibly other unrecognized conditions were met, ENSO-related processes seemed to have triggered cutting of new north channel outlets directly seaward across the ebb-edge spit. Table 3-2 lists 13 El Niño events identified and categorized by strength according to Quinn et al. (1978). All of the cycles (identified from the bathymetric charts, Appendix A) started within 12 months of the onset of an El Niño.

All but two of the ten El Niño-initiated channel reorientations between 1852 and 1998 were followed by a complete growth-rotation-dissection cycle. One of the two exceptions was in 1954 when a new outlet began during an El Niño, developed for 5 years, but then filled in, reconnecting the temporarily isolated distal shoal to the Cape Shoalwater spit. In 1959, when the 1954-iniated outlet closed, the next outlet was initiated by the next El Niño.

A strong correlation between timing of spit dissection and El Niño (Figure 3-13) suggests that a physical connection exists. Understanding this connection would make new channel alignments predictable. One cost-reducing opportunity provided by this predictability would be the scheduling of any channel relocation dredging (discussed further in the next section, Geomorphic Estimates of Shoaling Rates) to coincide with a predicted El Niño. A second and more active approach would be to investigate the practicality of engineering interventions mimicing the El Niño-related processes that restart channel cycles thereby creating new outlets needed for navigation ahead of the natural cycle.

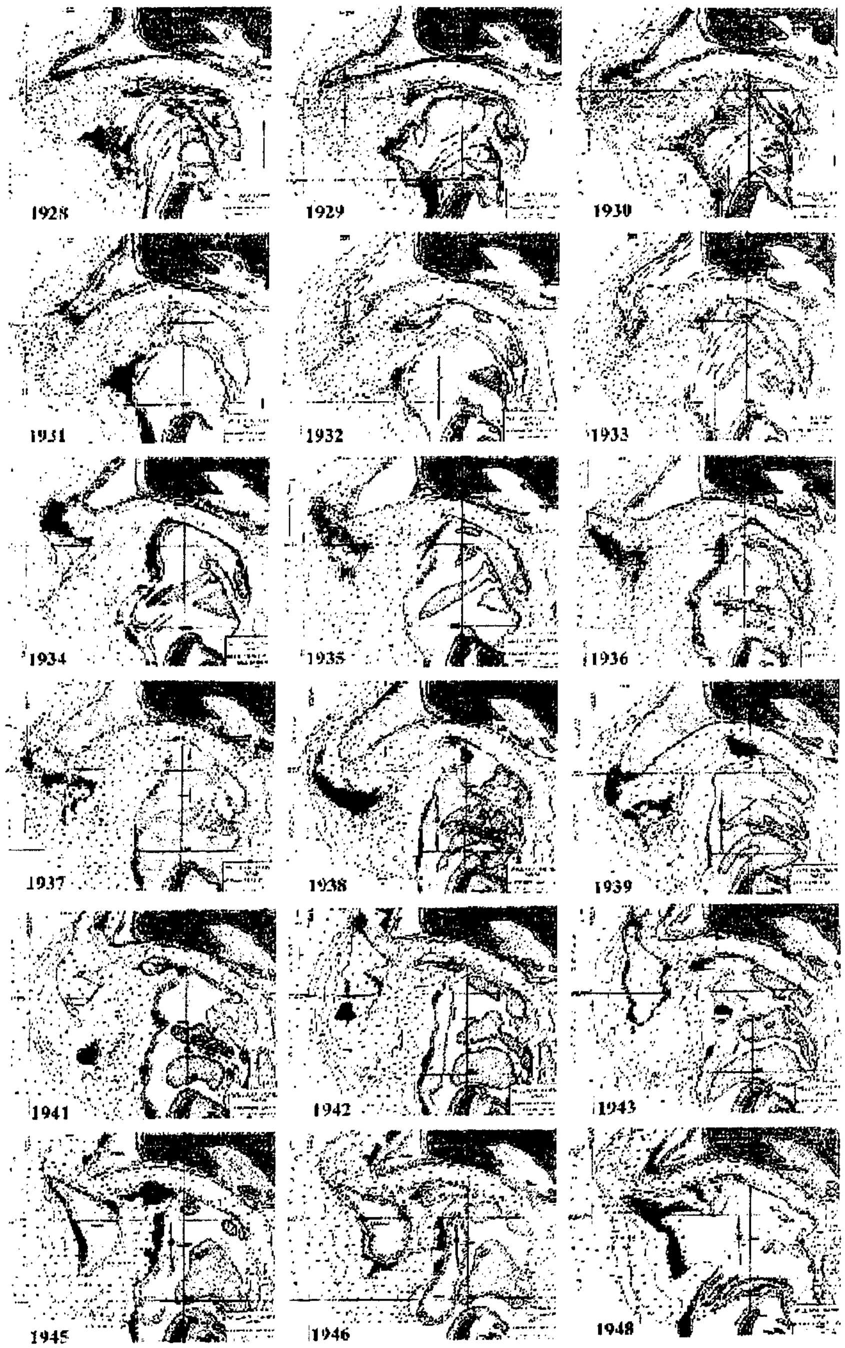


Figure 3-11. Twenty-year sequence illustrating two channel-spit cycles

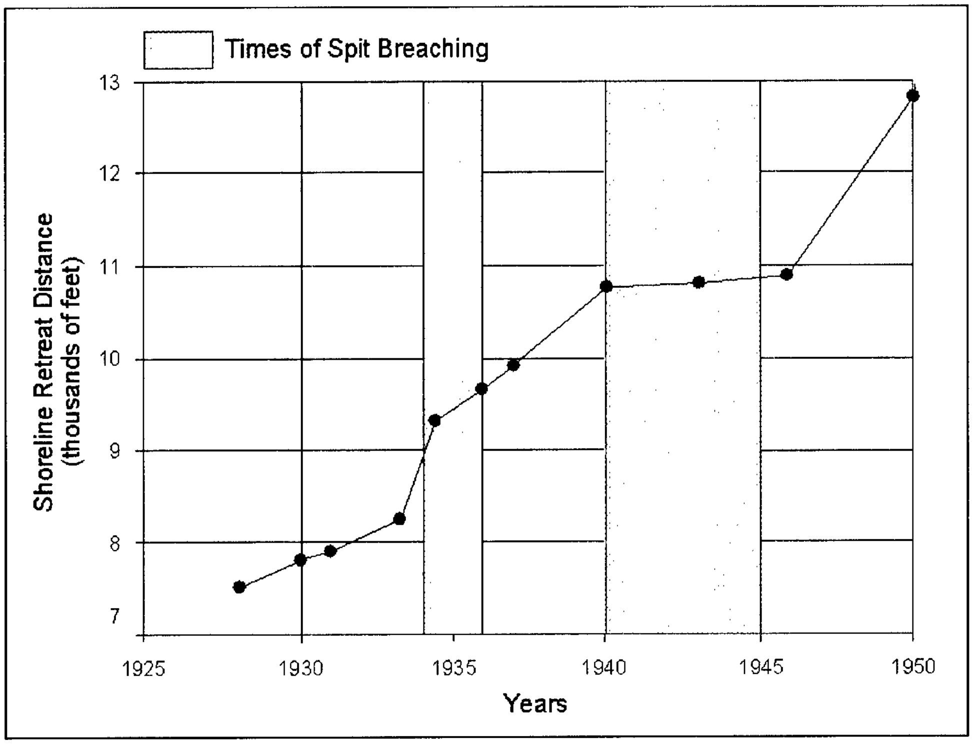


Figure 3-12. Rates of North Cove shore recession (data from Pacific International Engineering[PLLC], 1999) and episodes of Cape Shoalwater spit dissection or breaching (Table 3-6)

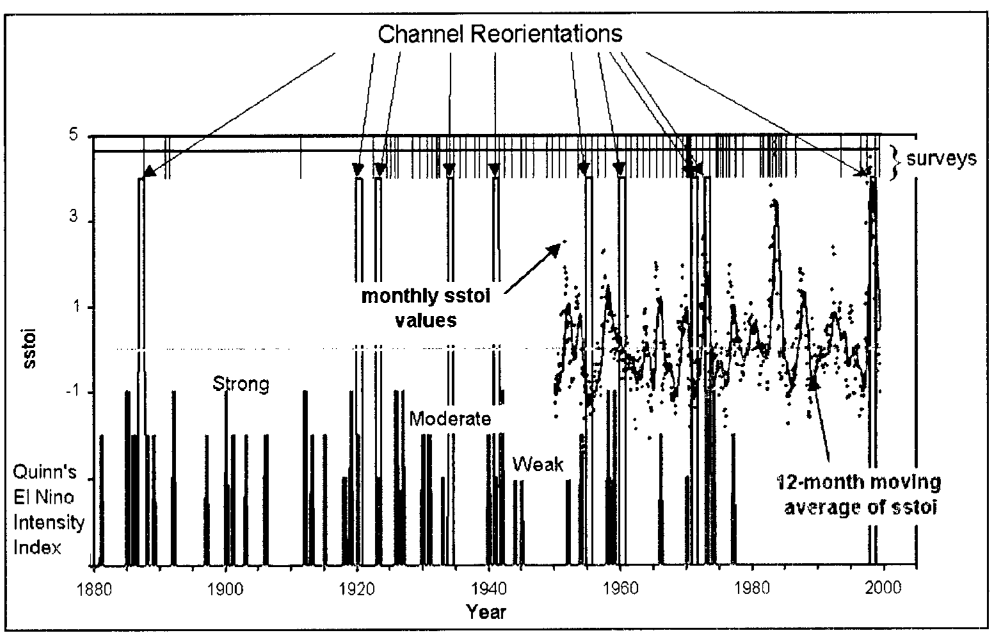


Figure 3-13. Timing of channel cycle with El Niño

Table 3-2
Timing of Post-1922 El Niños (from Quinn et al. 1978) and Restoration of New North Channel Outlets

Date	Event
1932-33	Weak El Niño
1933	Initiation of spit dissection leading to reorientation of primary North Channel outlet
1939-40	Moderate El Niño
1941-42	Strong El Niño
1941	Initiation of spit dissection leading to reorientation of primary North Channel outlet
1943-44	Weak El Niño
1951-52	Weak El Niño
1953-54	Moderate El Niño
1954	Initiation of spit dissection leading to reorientation of primary North Channel outlet
1957-58	Strong El Niño
1959	Shoaling seals outlet that was initiated in 1954
1960	Full spit dissection and reorientation of primary North Channel outlet after a temporary interruption in 1959
1965-66	Moderate El Niño
1969-70	Weak El Niño
1970	Initiation of spit dissection leading to reorientation of primary North Channel outlet
1972-73	Strong El Niño
1976-77	Moderate El Niño
1982-83	Strong El Niño
1997-98	Strong El Niño
1998	Iniation of spit dissection and reorientation of primary North Channel outlet

This study found evidence of repeated increases in monthly mean sea level on the order of 0.7 to 1 ft that last for about a month at nearby gages during the El Niños that coincided with new channel initiations. Because the bay tidal flats are so large, these higher water levels can significantly increase the tidal prism and the ebb discharge velocities. It can be speculated that increased momentum and higher velocity of tidal discharges over an elongate spit that is temporarily submerged an additional foot might promote cutting of a new outlet. The position of such a cut would be at the inflection point of the channel where momentum of the ebb flow would still be in the direction forced by orientation of the north bay shore. The alignment of the new cut would tend to parallel this shore-directed flow. These suppositions fit the characteristics of all ten new outlet initiations identified in Figure 3-13. There would also be, however, a tendency for the spatially averaged discharge velocity to decrease when higher water levels increased the flow cross-sectional area. Channelization of flow could also change and affect the location of peak discharge velocities. Evaluating which of these competing factors would dominate is beyond the present scope of work.

Monthly anomalies in sea surface temperature optimum interpolation indices (sstoi) were obtained from http://nic.fb4.noaa.gov/data/ cddb/cddv/-ssoti.indices. The buoy measurements of sea surface temperature selected for this study represent a region from 0 to 10 deg S and 80 to 90 deg W. This region was selected in a compromise between proximity to the North American Continent and longevity of the records, but data from other portions of the monitored equatorial area would probably show similar results. The anomalies are departures of the optimum interpolated temperature measurements from their base values for the period 1951-1979. A running average of these values is shown in Figure 3-13.

The timing of earlier El Niños was developed by Quinn et al. (1978). Quinn's index combines many factors and sources of information on global pressure systems and coastal South American rainfall, fisheries, sea surface temperatures, and sea bird reports dating back to the 1700s. Quinn's index is widely cited by climatologists and has been used in coastal investigations (e.g., Seymour 1998). It is adopted here to investigate the timing between El Niños and earlier episodes of channel reorientation.

Quinn did not attempt to specify the month of initiation or the duration of El Niños. A modification that uses bars to represent the time and intensity of Quinn's index is adapted in Figure 3-13. The bars are centered on 31 December because the typical El Niño causes warmest water temperatures in the tropical east Pacific around the end of the calendar year. Bar widths were arbitrarily set to span the six months from 1 October to 31 March. The actual onset and duration of El Niños vary as indicated by changes in peaks of the smoothed sstoi curve. Heights of the bars reflect Quinn's intensity categories. Pre-1980 gaps between the bars indicate low intensity El Niño-, neutral-, or La Niña-years. Note that where they overlap, all of Quinn's bars align with a peak in the sstoi index. All channel reorientations began during an El Niño.

This study also found evidence of repeated increases in month mean sea level on the order of 0.7 to 1 ft for several months during El Niño events that coincided with new channel initiation. Because tidal flats are so large, these higher water levels significantly increase the tidal prism and the ebb discharge velocities. It can be speculated that the increased momentum and higher velocity of tidal discharges over an elongate spit temporarily submerged an additional foot would promote cutting of a new outlet. The position of such a cut would be at the inflection point of the channel where momentum of the ebb flow would still be in the direction forced by orientation of the north bay shore. The alignment of the new cut would tend to parallel this shore-directed flow. These suppositions fit the characteristics of all ten new outlet initiations identified in Figure 3-13.

Storm paths across the north Pacific shift southward during El Niños. Seymour (1998); Inman, Jenkins, and Elwany (1996); and Komar (1986, 1997, and 1998) attribute dramatic increases in shore damage along the California and Oregon coasts to significantly higher waves coincident with the 1982 and 1998 El Niños. Changes in storm paths and frequency as well as increased wave heights and altered wave directions during El Niño are other potential factors that could alter erosion patterns at Willapa Bay and trigger new channel cycles. Relationships between these local processes and channel cycles have not been examined because of time constraints and specific objectives of the present study.

Geomorphic Estimates of Shoaling Rates

Approaches

In the northern half of the Willapa entrance, the general rate of long-term sediment transport from the north can be inferred from the gradual, but historically persistent southward growth of the Cape Shoalwater submerged spit (Figure 3-11 and Appendix A). For a coastal site in Alaska, Moore and Cole (1960) estimated sediment transport rates from the rates of spit growth. By adopting a similar approach, the rate of sediment influx to the Cape Shoalwater spit can be calculated from historical changes in bathymetry. This rate is used to infer a channel shoaling rate. In a second approach, shoaling rates for the Willapa Alternatives are estimated from historical shoaling rates in areas related to the channel templates. Templates are geometric forms that match the channel design cross section (width, depth, and side slopes). One or more template is associated with each channel Alternative. Volumes between templates and the entrance bathymetry were calculated for a number of years. By positioning templates in various locations, shoaling requirements were estimated for each alternative. Finally, annual bathymetric changes are combined with the calculated initial dredging requirements, the conclusions from spit growth measurements, the impacts of cycles, and differences in shoaling of migrating versus stationary channels to derive a 50-year dredging estimate.

These methods are approximate and depend on assumptions that greatly simplify the process of sediment transport, channel trapping, and variations in channel flushing potential. Other reasonable assumptions are possible. However, the degree of consistency among these geomorphically based estimates, comparison of their results with those from numeric simulations, and engineering experience all combine to support the methodology and to identify further work that would improve cost predictions for channel alternative.

Approach I: Estimates based on spit growth

Spit elongation. As shown in Figure 3-10 and 3-11, the channel and, therefore, the ocean bar migrate southward as the Cape Shoalwater spit progrades. Bar movement tracks with the southern terminus of the submerged spit. Superposing historical positions of the bar on a modern chart shows that the bar has moved across the whole entrance area at different times and therefore crosses all the channel alternatives (Figure 3-14). Not shown in Figure 3-14 is the continued southward migration of the distal portion of the spit, which carries the dissected shoal south of the entrance bar. Shoal migration was shown in Figures 3-10 and 3-11. Calculating the average growth rate of the spit gives an estimate of the volume of sand that must move over the area of any entrance channel (either as part of the growing spit or the dissected shoal).

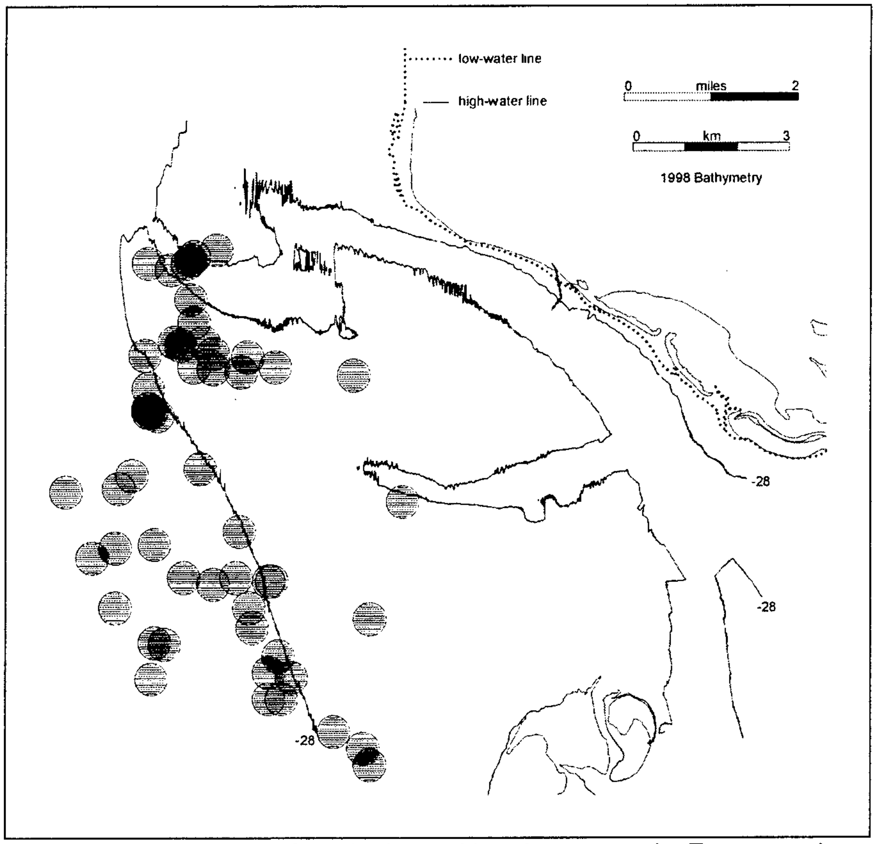


Figure 3-14. Positions of north channel bar (1911-1985). For general reference, shorelines and 28-ft depth contours are taken from 1996 aerials and the 1998 survey, respectively. The circles show that the terminus of the Cape Shoalwater spit reached positions that historically spread across the whole bay entrance. Circles indicate historical positions of the bar

Assumptions. Measuring spit growth and inferring transport rates depend on several assumptions that, although not strictly valid, are considered to be approximately correct. These assumptions are partially confirmed in this report through comparison of independent shoaling estimates from other approaches: spit growth, natural channel shoaling, and numerically simulated shoaling.

For sake of argument, begin by assuming spit elongation is fed by sand transported out of the north. Sand seems to move predominantly southward along the spit and accumulates in the deeper water off the south end of the spit. There are situations, in high-velocity steady flow, in which bed forms migrate in a direction opposite to sediment transport such as when sand moves downstream and settles on the upstream flank of the next antidune moving upstream in supercritical flow. At the Willapa entrance, flow across the channel is not supercritical, and the spit is large and slow moving. Therefore, the volume of sand that might be accreting near the southern tip of the spit from a source south of the proposed channels can be assumed to be volumetrically negligible. This assumption is not crucial, but simplifies application of the results because spit growth and/or migration would be driven by processes (including wave and tidal

current transport) that would carry the sand across any proposed channel location south of the dissection zone.

The last and most uncertain assumption in the spit growth approach is the percentage of the cross-channel transport that would accumulate in the channel from one year to the next. Some of the sand carried southward over the spit probably would not accumulate in the channel because it would be carried over, through, or seaward of the channel and continue downcoast. Such bypassing, if not accounted for, would contribute to an overestimation of shoaling by the spit growth estimates. On the other hand, a portion of the southward transport probably moves over the submerged spit without enlarging it, i.e., without settling on the south tip and contributing to its annual growth rate. This portion of the transport could still become trapped in a channel and therefore would contribute to an underestimation of channel shoaling by the spit growth approach. The volume bypassing the channel here tentatively assumed to be only negligibly larger than that bypassing the spit, i.e., transiting over the spit without contributing to its annual growth. Under these assumptions, overestimates approximately balance underestimates, and the growth rate of the spit approximates a background shoaling rate for any fixed channel across the Willapa Bay ebb shoal.

Resulting rates. Figure 3-15 shows three mature spits for which progressive southward growth is well documented by the charts. The volumes contained under the spit surfaces, above the plane of 20 ft (mllw) depth, and south of the line of dissection (approximately the proposed location for North Channel Alternative 3A) are shown in Figure 3-16. These volumes must equal the product of the average growth rate (on the vertical axis) times the duration of spit growth (segment on the horizontal axis). On this 1- to 2-decade scale, the average annual net rate of transport onto the spit varied between 1.5 and 3.1 million cu yd/year.

These derived rates are at the upper limit of reported longshore transport rates, but they are for an area where wave-driven longshore transport is supplemented by strong tidal transport in the bay entrance. The magnitudes seem appropriately large and are not inconsistent with the magnitude of maintenance dredging from the Columbia River and coastal estuaries in Oregon (U.S. Army Engineer District, Portland 1988). The large volumes of annual spit growth and their variations from one episode to the next are the first observations consistent with accepting the assumptions inherent in the spit growth approach to estimate shoaling rates. Another approach based upon annual bathymetric changes is discussed in the next section.

Approach II: Estimates using annual bathymetry changes

Presentation of 1998-99 changes across the entrance. The most detailed and accurate surveys to date of the Willapa Bay entrance are those of 1998 and 1999. Therefore, the best-known volume changes in the areas of the proposed channels are those for this recent period. The period 1998-99 also coincides with the most intense hydrodynamic measurements (Chapter 4), modeling of wind waves (Chapter 5), and modeling of circulation, and sediment transport (Chapter 6) conducted as part of this study. For these reasons, it will be valuable to examine the 1998-99 bathymetric changes in detail.

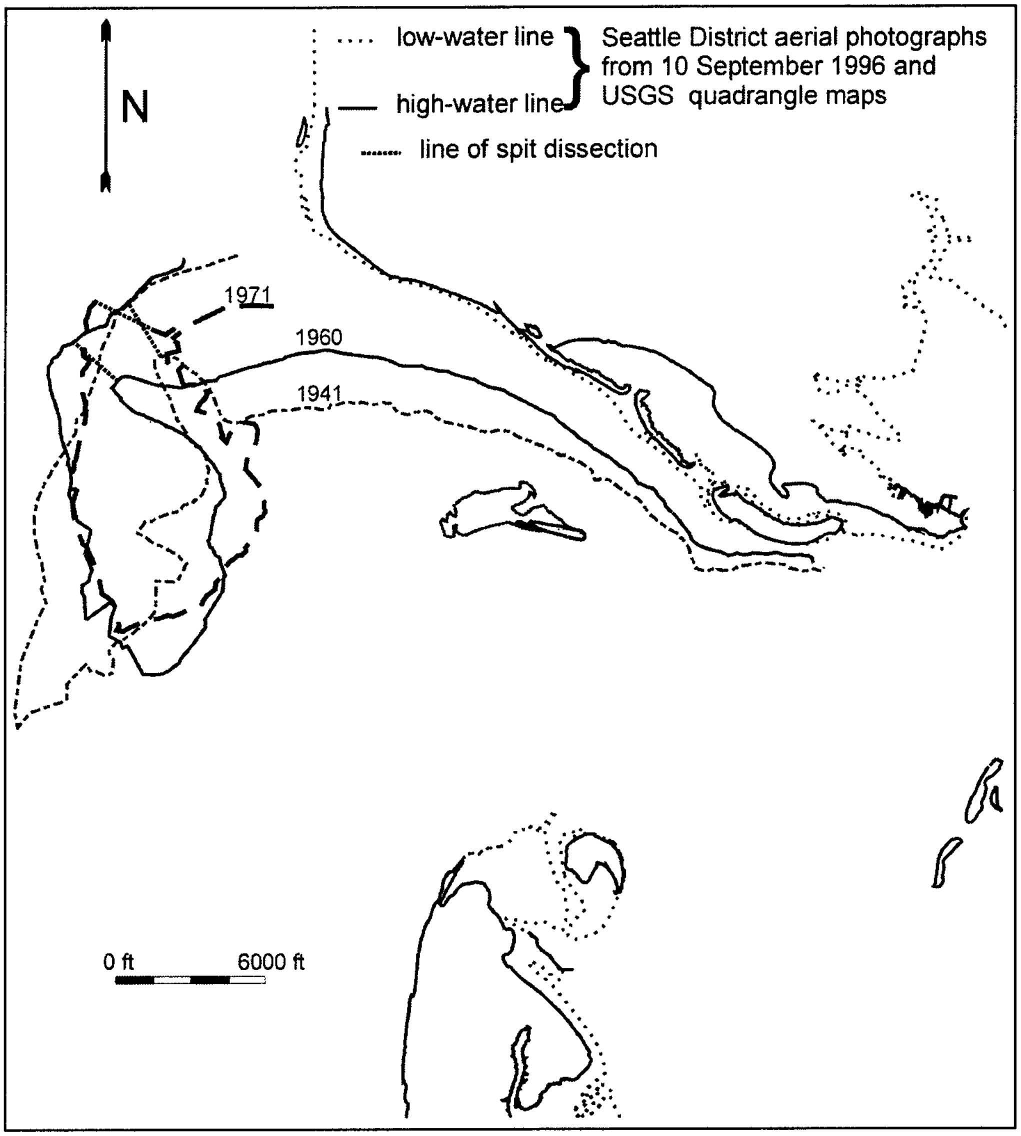


Figure 3-15. Area of spit above –20-ft-mllw elevation for three episodes for which the spit growth rate was calculated

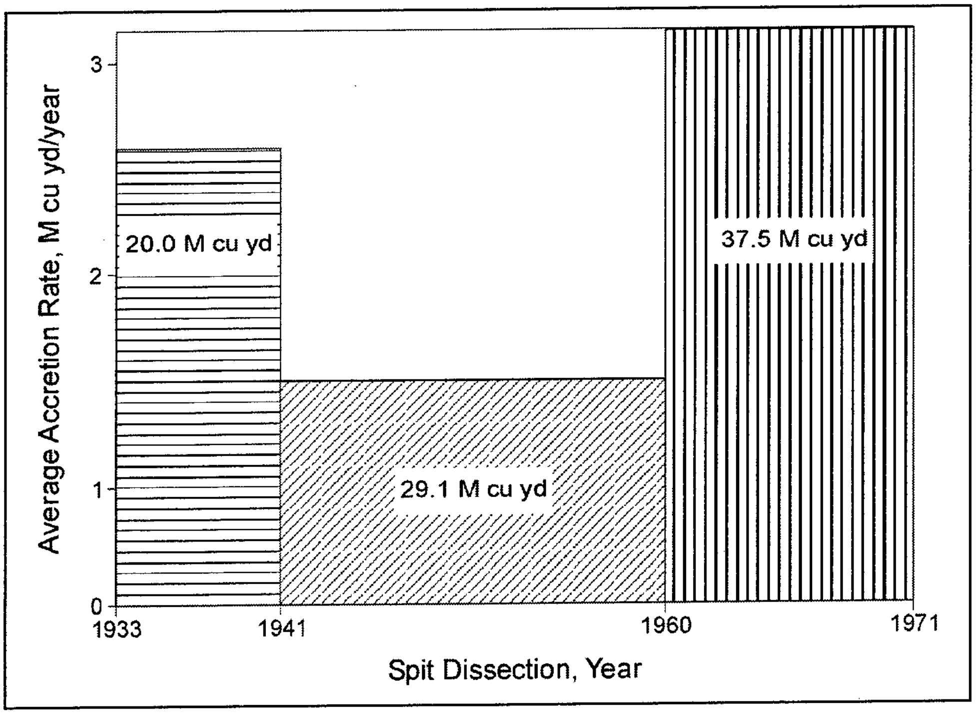


Figure 3-16. Long-term channel shoaling rate estimated from spit growth rates

The 1999 survey covered a much smaller area than the baywide survey of 1998. Interim results obtained in the winter of 1998 had focused attention on North and Middle Fairways. So, in the spring of 1999, instruments were moved to the northern half of the entrance and the hydrographic survey confined to this area. Over the 8.5-month period between these two surveys extensive and substantial erosion (>10 ft) occurred along inner and outer reaches of the North Channel. Substantial deposition (thicker than 10 ft) marked both banks of the inner North Channel and its outer S-curve. Enlargement of the outlet initiated during the 1997-98 El Niño had continued. Ocean waters moved more directly in and out of the north channel as the older outlet to the south shoaled. Shoaling was especially rapid in the north-to-south reach of this older outlet (Figure 3-17).

There appears to have been widespread, but thinner erosion over most of the offshore area beyond the channels. There also appears to have been substantial erosion along the north bank of the inner middle channel. These changes are probably more apparent than real. Although the offshore changes are widespread, vertical differences there are small; therefore, any datum error could have contributed to a relatively large part of this apparent erosion. The more substantial drop in elevations along the inside of the middle channel could be largely an artifact due to the high resolution of depths around Sand Island by the SHOALS system in 1998 and the confinement of 1999 soundings to the nearby deeper channel. Both of these uncertainties are avoided by the methodology adopted here to estimate channel shoaling.

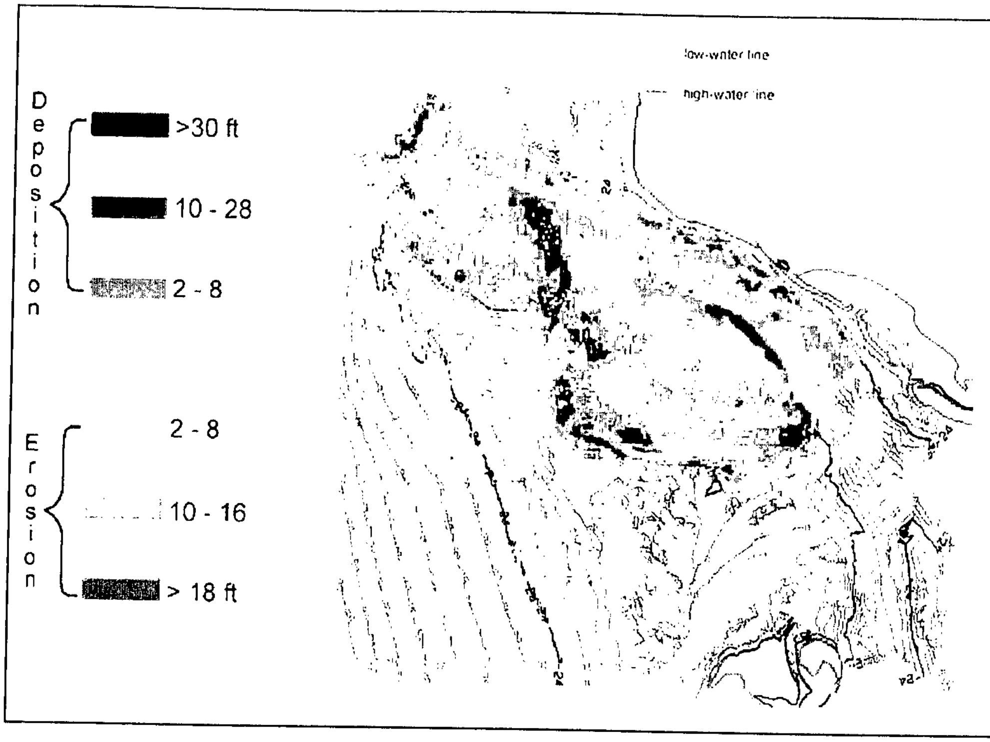


Figure 3-17. Differences between the 1998 and 1999 surveys

The 1998-99 period, as any other year, is only a snapshot in the evolution of a navigation channel. Rates of erosion (like the winds, waves, and currents that drive sediment transport) are extremely variable. Long-term trends seen in the channel cycle further compound the danger of extrapolating from changes measured in any single year. Integration of 1998-99 changes presented in this section into a longer term picture are dealt with later in this chapter.

Presentation of 1998-99 in channel templates. Soundings were conducted over an 87-day period in the summer of 1998. The areas of the Alternative Channels (used in the approach of this section to shoaling estimates) were surveyed over a shorter period in August of that year (Figure 3-2). The area of Alternatives was resurveyed on 28 and 29 April 1999 except for five supplemental lines added on 27 May. Differences between the 1998 and 1999 surveys capture the net change over an 8.5-month period.

A template is a geometric form having the dimensions of the design channels. There is a channel template for each Alternative. Alternatives that differ only in regard to design of training structures share the same template. Volume changes were quantified by calculating volume differences within each of seven distinct channel templates associated with the Alternatives that passed screening (columns 2-4 in Table 3-3), i.e., changes outside the sloping sides of the templates or beneath their base were not included in this analysis.

In the right-hand portion of Table 3-3, volumes are reexpressed as annual change rates (obtained by dividing 8.5/12 into the volume measurement). The volumes are also reexpressed as annual changes in elevations averaged over the areas of the templates. Changes in elevation are easily visualized but should be

considered cautiously, bearing in mind the large area covered by the templates. Templates are close to 6 miles in length and include sizable areas of negligible or no change that reduce the impact of the large changes in elevation that were confined to more restricted areas.

In some areas, the templates crossed regions where the natural depths were uniformly below the template. Over these areas, by the adopted methodology, cuts and fill are zeroed because they would not affect navigation. It could be argued that sedimentation below the design depth might eventually raise the bottom above the channel design depth and, therefore, indicate greater shoaling potential for that particular channel alternative. This argument is speculative and disregards the observation that the bottom had not reached the design elevation in 1999. Therefore, it is appropriate to exclude changes below the design depth just as it is to exclude changes laterally outside the templates.

If the bar channel had been maintained in 1999, the volumes that would have needed dredging can be judged from either the net volume differences or the volume of fill that accumulated within each template. Net and fill volumes are given in Table 3-3. To compare modeled bathymetric changes to measured changes, it may be more informative to look at cut and fill separately.

As indicated by the predominance of erosion (negative changes) in Table 3-3, none of the alternatives would have required much dredging in 1999. The only net accumulations were in templates 3B and 3G. These accumulations should not be mistaken as indicating any requirement to dredge Alternatives 3B or 3G. Templates 3B and 3G would not be maintained by dredging. These templates represent only the most extreme acceptable position of migration for the North Channel. Before dredging in 1999, it would have been apparent that dredging operations needed to be relocated to expand newly formed northern outlets. In considering relative dredging requirements for each alternative, the implications for Alternative 3A and 3B would be identical for 1999, as would be the implications for 3F and 3G. None of the alternatives would have had unacceptable dredging requirements in 1999. What, however, do the long-term statistics imply?

Historical changes over templates. The magnitude of annual sedimentation can be estimated by averaging year-to-year differences over a long period. Two weaknesses are acknowledged. First, the design channels, being deeper than natural depths, will act as sediment traps and accumulate more sediment than they would without dredging. There would be no significant increase in tidal flushing following the increase in channel cross section because the bay already completely fills during normal tidal cycles as shown in Chapter 5. Therefore, no reduction is expected in sediment trapping. The second neglected aspect in the annual bathymetry approach is that historical data are probably insufficient to resolve differences by comparing changes under one design template with those under another. Soundings were often sparse over the 500-ft-wide templates. The Seattle District had little need historically to survey details away from the natural channels. Large areas over the templates were dry land during many of the earlier surveys. The annual bathymetry approach will therefore establish differences among the alternatives by assigning them to certain classes rather than examining historical changes within each Alternative template.

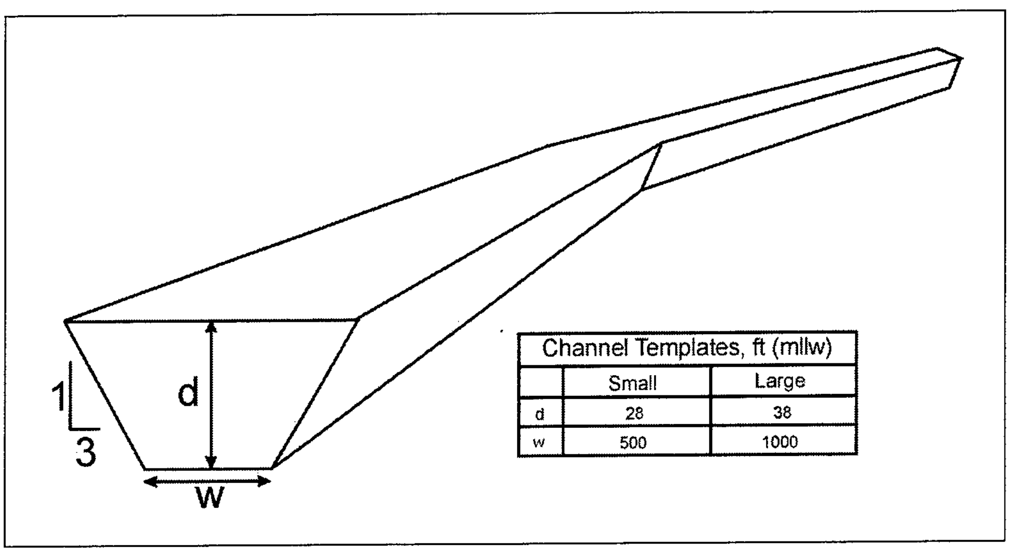
Table 3-3
1998 to 1999 Bathymetric Changes Within Templates

Design Depth is	8-Month Change 8-98 to 4-99			Annualized Change Rate			
				Fill		Net Change	
-28 ft mllw	Fill cu yd	Cut cu yd	Net Change cu yd	cu yd	cf/sq ft	cu yd	cf/sq ft
Channel Template 3A							
Template Area, sq ft	17,168,038						
Above Design Depth	3.7E+04	-8.0E+05	-7.6E+05	5.2E+04	0.1	-1.1E+06	-1.7
Below Design Depth	1.4E+05	-3.8E+06	-3.7E+06				
Total	1.7E+05	-4.6E+06	-4.5E+06	2.5E+05	0.4	-6.3E+06	-9.9
Channel Template 3B							
Template Area, sq ft	33,155,116						
Above Design Depth	1.0E+06	-3.4E+03	1.0E+06	1.4E+06	1.2	1.4E+06	1.2
Below Design Depth	2.4E+06	-2.0E+06	4.3E+05				
Total	3.4E+06	-2.0E+06	1.4E+06	4.9E+06	1.1	2.0E+06	1.6
Channel Template 3H							
Template Area, sq ft	17,061,317						
Above Design Depth	5.7E+05	-1.0E+06	-4.5E+05	8.0E+05	1.3	-6.3E+05	-1.0
Below Design Depth	1.1E+05	-1.7E+06	-1.5E+06				
Total	6.8E+05	-2.7E+06	-2.0E+06	9.6E+05	1.5	-2.8E+06	-4.4
Channel Template 4A							
Template Area, sq ft	11,923,267						
Above Design Depth	2.9E+05	-2.4E+05	5.1E+04	4.1E+05	0.9	7.2E+04	0.2
Below Design Depth	4.0E+05	-4.5E+05	-5.3E+04				
Total	6.9E+05	-6.9E+05	-1.7E+03	9.7E+05	2.2	-2.4E+03	0.0
Channel Template 3F							
Template Area, sq ft	24,499,897						
Above Design Depth	3.1E+05	-3.2E+06	-2.9E+06	4.3E+05	0.5	-4.1E+06	-4.6
Below Design Depth	1.3E+05	-2.3E+06	-2.2E+06				
Total	4.3E+05	-5.5E+06	-5.1E+06	6.1E+05	0.7	-7.2E+06	-8.0
Channel Template 3G							
Template Area, sq ft	43,596,918						
Above Design Depth	3.5E+06	-7.5E+05	2.7E+06	4.9E+06	3.0	3.9E+06	2.4
Below Design Depth	3.7E+05	-3.3E+06	-2.9E+06				
Total	3.8E+06	-4.0E+06	-1.6E+05	5.4E+06	3.4	-2.3E+05	-0.1
Channel Template 4E							
Template Area, sq ft	39,797,130						
Above Design Depth	1.6E+06	-1.7E+06	-9.0E+04	2.2E+06	1.5	-1.3E+05	-0.1
Below Design Depth	4.1E+05	-1.1E+06	-6.8E+05				
Total	2.0E+06	-2.8E+06	-7.7E+05	2.8E+06	1.9	-1.1E+06	-0.7

Values "Above Design Depth" are related to maintenance dredging.
Values "Below Design Depth" are for model corroboration.

Size class. Larger channels have greater capacity for shoaling. All Willapa Channel Alternatives fall into one of two size classes depending on the cross section (Figure 3-18). Although not of identical lengths, all the North Channel templates extend from their respective design depths on the ocean ends to a common area near the region of the SR-105 project so that the areas are roughly compatible in terms of comparative shoaling volumes and locations (Figure 3-19). Little shoaling occurred toward the landward end of the North Channel templates because the North Channel is naturally wide and deep in this region. Middle Channel Alternatives were extended over sufficient lengths to encompass all bathymetric changes that occurred above design depths. Bottom changes beyond the 1V:3H side slopes of the templates or below their 28- and 38-ft design depths will be zeroed for reasons discussed previously.

The total fill volume within each channel template was obtained as the sum of the volume differences in all the areas where the terrain model for 1999 was above that for 1998 (Column 2 of Table 3-3). The calculated cut volume reported in Column 3 is the negative of the sum of the volumes from all the areas where the 1999 model was below that of 1998. The net change (Column 3) is the algebraic sum of cut and fill. Results are summarized in Table 3-3 and given more fully in Appendix B. Erosion prevailed, but Figures 3-20 and 3-21 show the spatial isolated areas of significant shoaling inside the templates.



Channel Templates, ft (mllw)		
	Small	Large
d	28	38
w	500	1000

Figure 3-18. Two channel cross-sectional designs

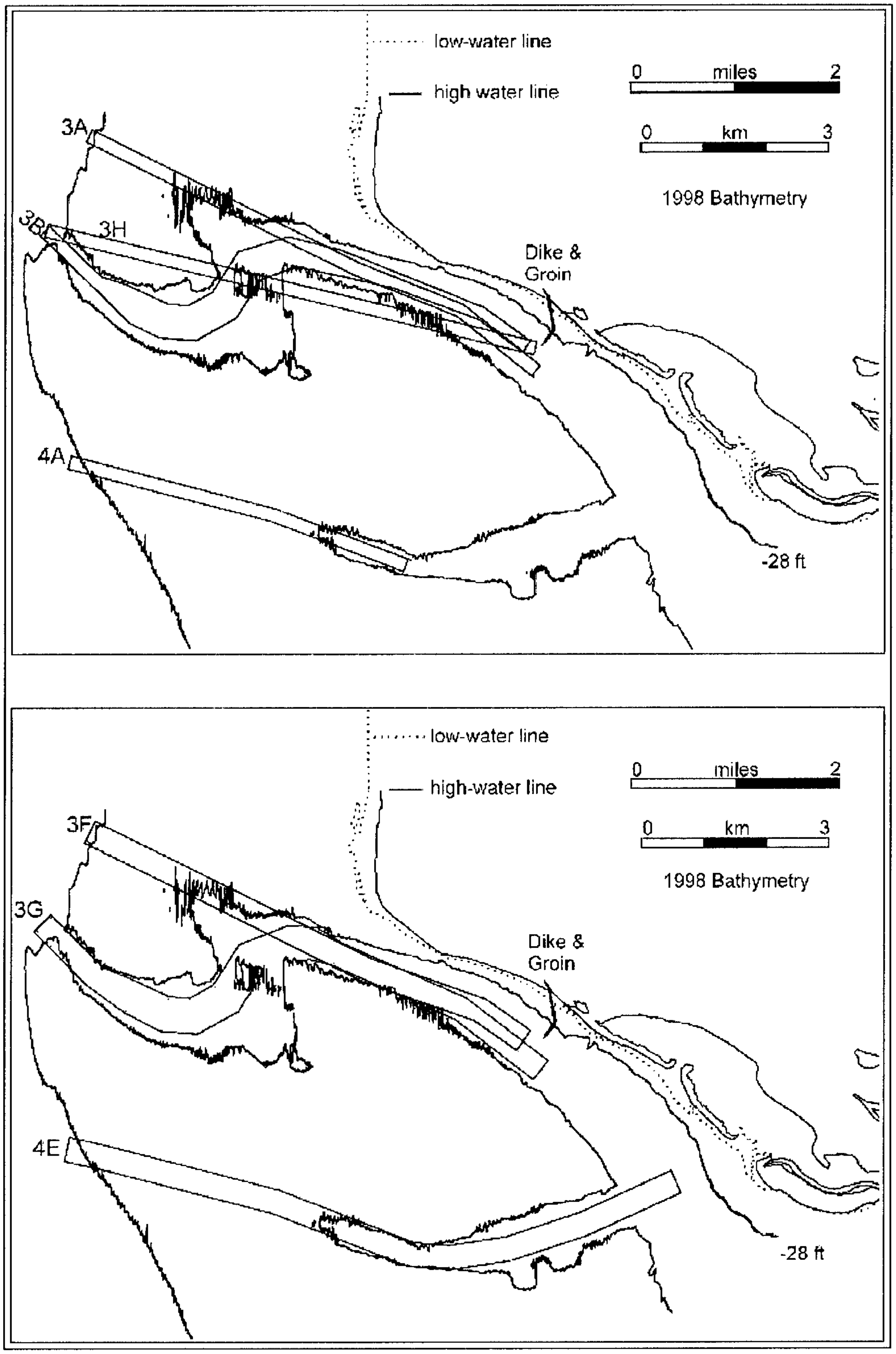


Figure 3-19. Seven channel alternatives for which geomorphic-based estimates were made of shoaling rates. Top panel shows 500-ft-wide, 28-ft-deep (mllw) alternatives and bottom panel shows 1,000-ft-wide, 38-ft-deep alternatives

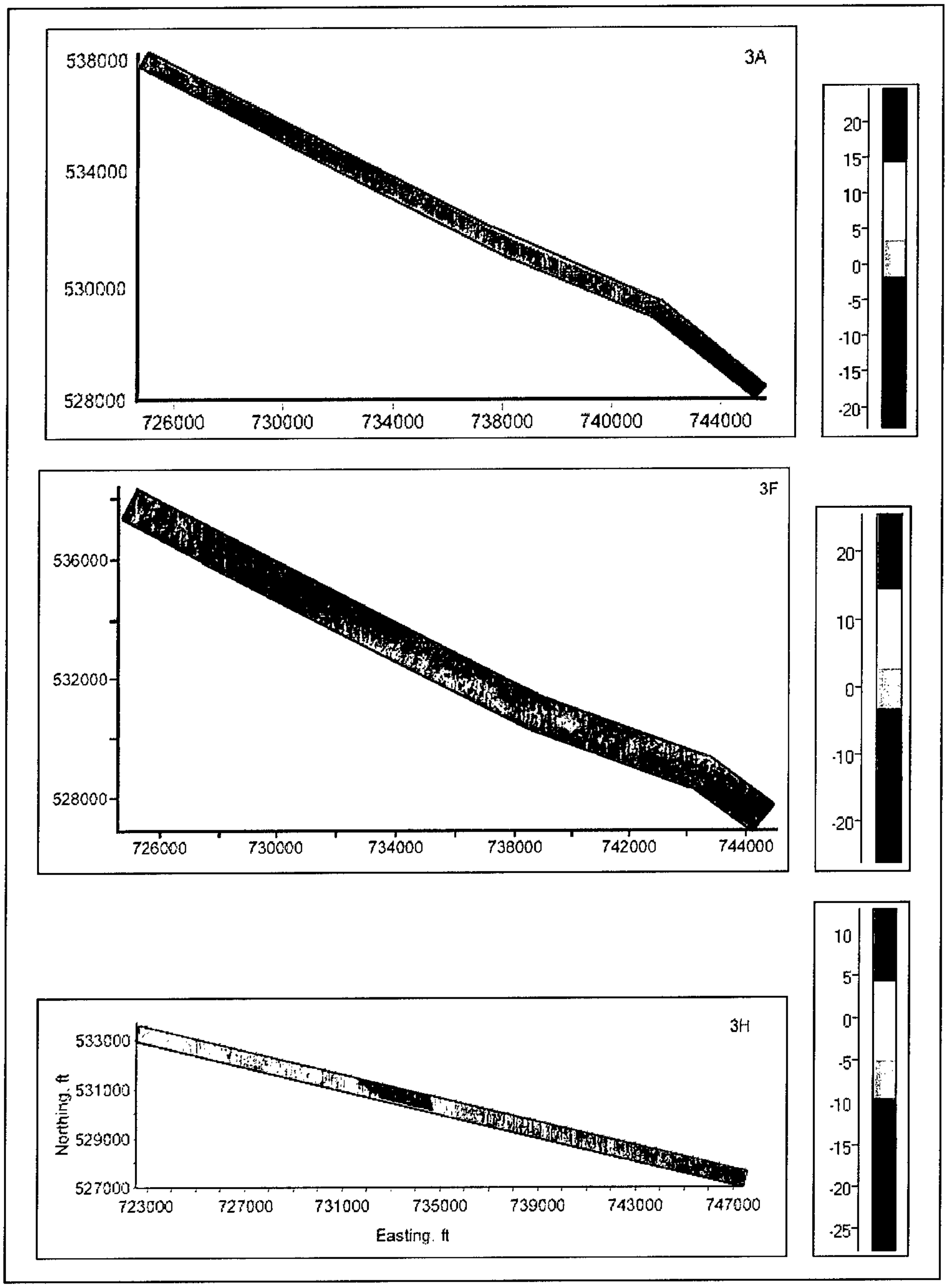


Figure 3-20. Shoaling and deepening in the shallower alternatives, 1998-1999. Note: shoaling was confined to a small area of the outer bar in Alternative 3A

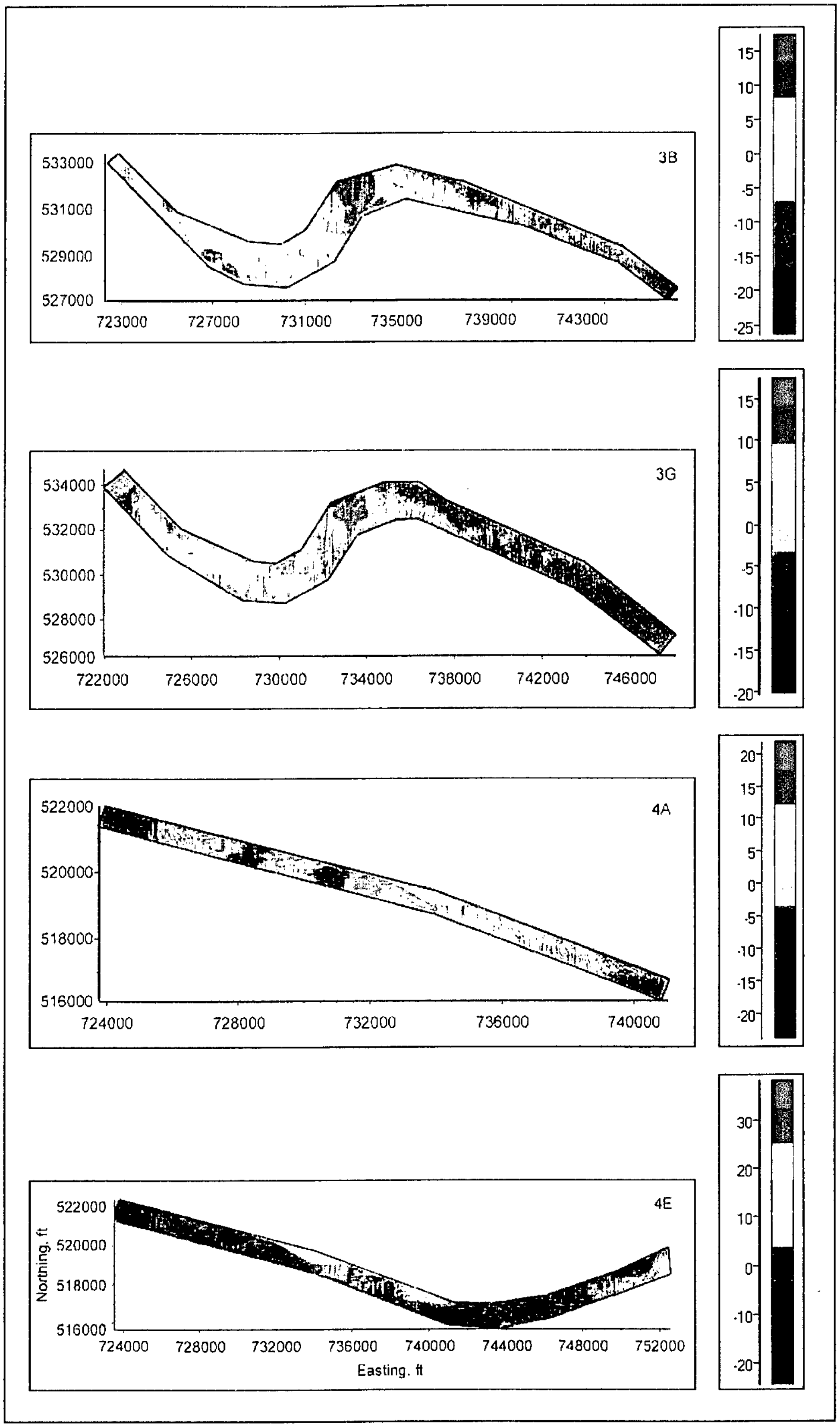


Figure 3-21. Shoaling in the deeper alternatives, 1998-1999

Dredging requirements are less if the maintained channel is allowed to migrate from year to year in a manner that takes advantage of natural channel migration. The Willapa Channel Alternatives fall into one of two classes based on position: stationary or migrating. Alternative 3B is in the migrating class. Positions of 3B migrate between positions represented by the 3A and 3B templates in Figure 3-20. Alternative 3F allows the channel to migrate between positions represented by Templates 3F and 3G. The other Alternatives are in the stationary class, i.e., dredging would maintain a fixed channel location from year to year as the entrance bathymetry and natural channels evolved around the navigation channel.

Table 3-3 indicates the savings obtainable from the migrating channel design. During the interval 1998-99, no material shoaled in the 3A template. By the assumptions of Approach II, it is inferred that no maintenance would have been required in 1999 if 3A had been the maintained channel in 1998. For the same period, 1 million cu yd shoaled in Template 3B. Alternative 3B is a migrating design, so maintenance is not inferred from this 1-million-cu-yd shoaling in a fixed geodetic template, but from shoaling that would have occurred along the migrating thalweg as long as the channel met navigation requirements. For 1999, Alternative 3B would have required moving the authorized channel (and buoy markers) to the newly opened northwest outlet in the position of the 3A template. So, from the measurements (Table 3-3) zero maintenance is inferred for both the stationary 3A and the migrating 3B. Likewise, zero maintenance is inferred from the 1998-99 changes in the wider migrating Alternative 3G even though nearly 3 million cu yd accumulated in the S-curve that 3G occupied in 1998 (compare net changes, Table 3-3).

Similar comparisons with the historical data would not be particularly useful. Earlier soundings were too sparse or did not cover template areas, and the variations in historical shoaling from year to year are noisy compared with spatial differences between alternatives. Therefore, historical volume changes have been calculated where the data are more suitable and used to estimate shoaling by channel classes.

Historical changes over data-dense areas not congruent with Alternatives. The data are suitably dense in time for the dates shown in Table 3-4 and dense spatially for the areas shown in Figure 3-22. The templates shown in Figure 3-22 are not the templates for the Channel Alternatives, but are thought to provide better historical data that are used to evaluate classes of Alternatives. Concentrating on fill that accumulated between surveys as most important for maintenance dredging estimates, the historical fill rates usually are in the range of 1-3 million cu yd/year. Assuming that the 16 cases in Table 3-4 are equally relevant to any of the Channel Alternatives, the average annual shoaling of 2.5 million cu yd/year can be adopted as an approximate background level of shoaling for any fixed channel across the northern part of the bay entrance.

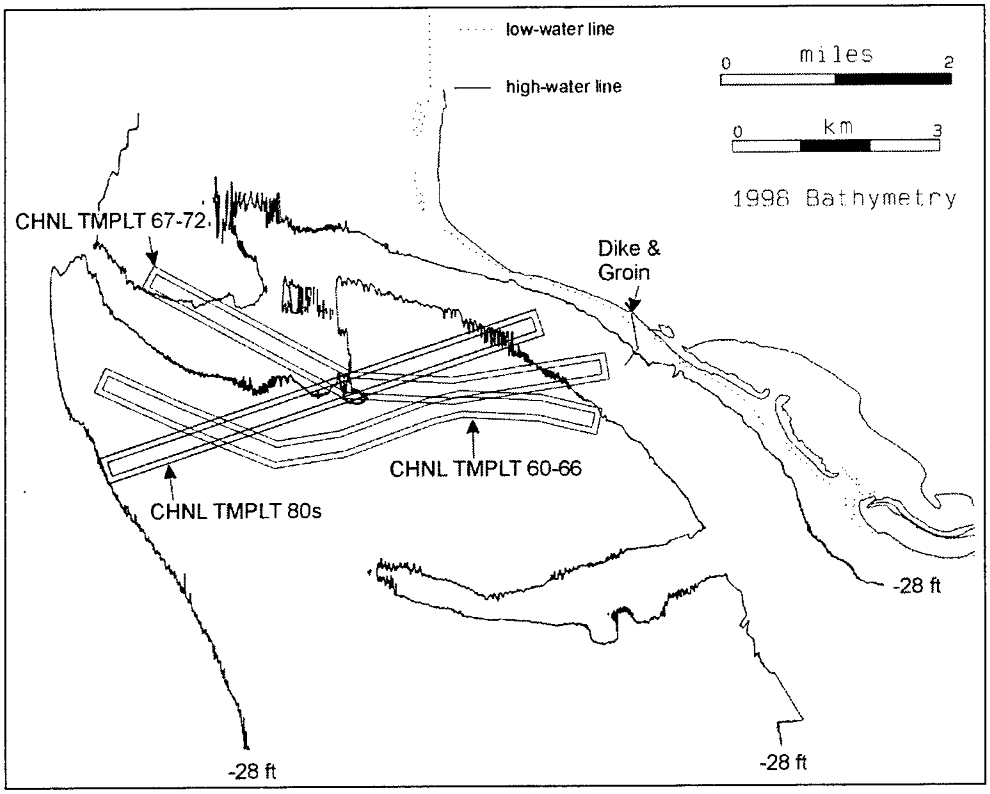


Figure 3-22. Additional areas in which shoaling statistics were calculated (contour elevation is with respect to mllw)

Size of template. The data from Tables 3-3 and 3-4 also verify the concept that the potential for shoaling goes up with increased cross section. Prior navigation studies have proposed methods for estimating shoaling that depend only on examination of historical bathymetric surveys of existing channels. For constant channel widths, Vincente and Uva (1984) proposed that the shoaling rates remain proportional to the difference between the actual channel depth and an equilibrium depth. The equilibrium depth could be determined by curve fitting to historical depth changes. Trawle and Herbich (1980) proposed a simpler relationship in which the percent increase in shoaling is equal to the percent increase in channel volume. Rather than adopt one of these empirical methods, the Willapa bathymetry provides a direct relationship that is expected to produce more accurate estimates. Note that in Figure 3-23 a nearly linear relationship exists between bathymetric changes in the large and small templates at Willapa. This relationship will be used in estimating shoaling for various classes of Alternative Channels

Stationary versus migrating Alternatives. Position is one of the classes used to estimate shoaling by the bathymetric change approach. A much smaller background shoaling rate is appropriate for migrating channels. The approximate background level for Alternatives 3B and 3G will be based on the rates measured in 1999 and on the observation that bar depth and width have always remained fairly steady in the North Fairway except during years when outlets are either growing or being sealed off in response to initiation of a new outlet to the northwest across the spit.

Resulting rates. Table 3-5 summarizes 1998-99 volume changes in ways that will be input to a lifetime shoaling rate. Net differences between these two particular surveys indicate widespread erosion. These "negative shoaling" rates are not considered typical, but several relationships based on the historical volume changes still apply to this atypical year. The magnitude of fills is similar. The linear relationship between volumes trapped in 28- and 38-ft-deep templates also agrees with that obtained using historic data.

Table 3-4 provides the statistics for estimating a background shoaling that would apply to any channel over the long term. This background shoaling rate exceeds the historical dredging rates (Table 2-1). Geomorphic methods are inexact, but the difference between past dredging volumes and estimated future requirements seems reasonable because past practice did not maintain the presently proposed channel dimensions.

Approach III: Lifetime dredging requirements

Lifetime dredging estimates are helpful for developing relative costs of the different Alternatives. For the stationary Alternatives, the lifetime requirements are simply the initial dredging costs plus the sum of annual maintenance costs over the lifetime of the project. Both components can be approximated well enough so that at least the relative ranking of stationary Channel Alternatives is robust. The migrating Alternatives require more consideration.

Table 3-4
Historical Volume Differences Within Templates in Data-Dense Areas

Changes Under Various Channel Footprints					Annualized Change Rate			
					Fill		Net Change	
Design Depths Referenced to mllw	Area sq ft	Fill cu yd	Cut cu yd	Net Change cu yd	cu yd	cf/sq ft	cu yd	cf/sq ft
CHN TMPLT 60-66								
July 1961 – October 1960		**Elapsed Time, years = 0.75**						
Above Design Depth (28 ft)	1.6E+07	4.0E+05	-1.7E+06	-1.3E+06	5.3E+05	0.9	-1.7E+06	-3.0
Above Design Depth (38 ft)	2.9E+07	3.8E+06	-3.2E+06	5.9E+05	5.1E+06	4.6	7.8E+05	0.7
July 1962 – July 1961		**Elapse Time, years = 1.00**						
Above Design Depth (28 ft)	1.6E+07	3.9E+05	-1.0E+06	-6.3E+05	3.9E+05	0.7	-6.3E+05	-1.1
Above Design Depth (38 ft)	2.9E+07	9.4E+05	-3.1E+06	-2.2E+06	9.4E+05	0.9	-2.2E+06	-2.0
July 1963 – July 1962		**Elapse Time, years = 1.00**						
Above Design Depth (28 ft)	1.6E+07	4.5E+05	-4.3E+05	2.5E+04	4.5E+05	0.8	2.5E+04	0.0
Above Design Depth (38 ft)	2.9E+07	1.8E+06	-1.5E+06	2.7E+05	1.8E+06	1.7	2.7E+05	0.3
July 1964 – July 1963		**Elapse Time, years = 1.00**						
Above Design Depth (28 ft)	1.6E+07	5.1E+05	-2.5E+05	2.6E+05	5.1E+05	0.9	2.6E+05	0.4
Above Design Depth (38 ft)	2.9E+07	2.0E+06	-6.8E+05	1.3E+06	2.0E+06	1.8	1.3E+06	1.2
August 1965 – July 1964		**Elapse Time, years = 1.08**						
Above Design Depth (28 ft)	1.6E+07	8.3E+05	-4.1E+05	4.2E+05	7.6E+05	1.3	3.9E+05	0.7
Above Design Depth (38 ft)	2.9E+07	2.2E+06	-2.0E+06	1.8E+05	2.0E+06	1.8	1.6E+05	0.1
								(Continued)

Table 3-4 (Concluded)

Changes Under Various Channel Footprints					Annualized Change Rate			
					Fill		Net Change	
Design Depths Referenced to mllw	Area sq ft	Fill cu yd	Cut cu yd	Net Change cu yd	cu yd	cf/sq ft	cu yd	cf/sq ft
CHN TMPLT 60-66								
July 1966 – August 1965	**Elapse Time, years = 0.92**							
Above Design Depth (28 ft)	1.6E+07	2.4E+06	-3.0E+05	2.1E+06	2.7E+06	4.6	2.3E+06	4.0
Above Design Depth (38 ft)	2.9E+07	6.4E+06	-7.1E+05	5.7E+06	7.0E+06	6.4	6.2E+06	5.7
CHN TMPLT 67-72								
August 1967 – July 1966	**Elapse Time, years = 1.08**							
Above Design Depth (28 ft)	1.5E+07	6.5E+05	-3.8E+05	2.7E+05	6.0E+05	1.1	2.5E+05	0.5
Above Design Depth (38 ft)	2.7E+07	1.9E+06	-1.1E+06	7.8E+05	1.7E+06	1.7	7.2E+05	0.7
July 1968-August 1967	**Elapse Time, years = 0.92**							
Above Design Depth (28 ft)	1.5E+07	7.8E+05	-1.8E+05	5.9E+05	8.5E+05	1.6	6.5E+05	1.2
Above Design Depth (38 ft)	2.7E+07	2.9E+06	-3.7E+05	2.5E+06	3.1E+06	3.1	2.7E+06	2.7
July 1969 –July 1968	**Elapse Time, years = 1.00**							
Above Design Depth (28 ft)	1.5E+07	1.6E+06	-1.1E+05	1.4E+06	1.6E+06	2.9	1.4E+06	2.7
Above Design Depth (38 ft)	2.7E+07	3.0E+06	-6.8E+05	2.4E+06	3.0E+06	3.0	2.4E+06	2.3
July 1970 – July 1969	**Elapse Time, years = 1.00**							
Above Design Depth (28 ft)	1.5E+07	7.6E+05	-7.5E+05	1.6E+04	7.6E+05	1.4	1.6E+04	0.0
Above Design Depth (38 ft)	2.7E+07	1.9E+06	-1.7E+06	2.1E+05	1.9E+06	1.9	2.1E+05	0.2
July 1971 – July 1970	**Elapse Time, years = 1.00**							
Above Design Depth (28 ft)	1.5E+07	7.7E+05	-6.5E+05	1.2E+05	7.7E+05	1.4	1.2E+05	0.2
Above Design Depth (38 ft)	2.7E+07	1.8E+06	-1.6E+06	2.0E+05	1.8E+06	1.7	2.0E+05	0.2
July 1972 – July 1971	**Elapse Time, years = 1.00**							
Above Design Depth (28 ft)	1.5E+07	9.0E+05	-3.5E+06	-2.6E+06	9.0E+05	1.7	-2.6E+06	-4.8
Above Design Depth (38 ft)	2.7E+07	2.3E+06	-7.2E+06	-4.9E+06	2.3E+06	2.2	-4.9E+06	-4.9
August 1973 - July1972	**Elapse Time, years = 1.08**							
Above Design Depth (28 ft)	1.5E+07	5.7E+05	-1.8E+06	-1.2E+06	5.3E+05	1.0	-1.1E+06	-2.0
Above Design Depth (38 ft)	2.7E+07	2.0E+06	-4.0E+06	-1.9E+06	1.9E+06	1.9	-1.8E+06	-1.8
CHN TMPLT 80s								
July 1981 – April & May 1981	**Elapse Time, years = 0.21**							
Above Design Depth (28 ft)	1.4E+07	1.6E+05	-7.4E+05	-5.8E+05	7.8E+05	1.5	-2.8E+06	-5.5
Above Design Depth (38 ft)	2.6E+07	1.2E+06	-2.6E+06	-1.4E+06	5.8E+06	6.0	-6.7E+06	-7.0
August 1982 - July 1981	**Elapse Time, years = 1.08**							
Above Design Depth (28 ft)	1.4E+07	3.0E+05	-5.2E+05	-2.2E+05	2.8E+05	0.5	-2.1E+05	-0.4
Above Design Depth (38 ft)	2.6E+07	8.6E+05	-2.5E+06	-1.7E+06	8.0E+05	0.8	-1.5E+06	-1.6
July & August 1983 - August 1982	**Elapse Time, years = 1.00**							
Above Design Depth (28 ft)	1.4E+07	4.6E+04	-5.6E+05	-5.2E+05	4.6E+04	0.1	-5.2E+05	-1.0
Above Design Depth (38 ft)	2.6E+07	3.5E+05	-1.7E+06	-1.4E+06	3.5E+05	0.4	-1.4E+06	-1.4

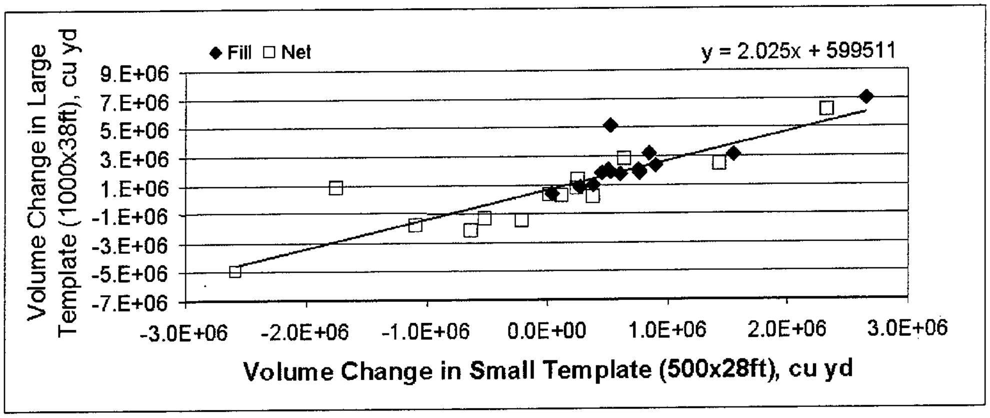


Figure 3-23. Relative shoaling rates for the two template cross sections

Table 3-5
Summary Of Initial Dredging and Volume Changes in Templates, 1998-1999

Template Description		Initial Cut cu yd	1998-1999 Survey Changes, cu yd		
			Cut	Fill	Net
3A*	North Channel Straight	7.60E+05	-8.00E+05	3.70E+04	-7.60E+05
3B*	North Channel 1,500 ft on curves	4.30E+04	-4.20E+03	1.00E+06	1.00E+06
3D	Raise SR-105 Dike				
3E	North Channel with Jetty				
3F*	3A, but 38- x 1,000-ft channel dimensions	5.70E+06	-4.20E+06	3.40E+05	-3.80E+06
3G*	3B, but 38- x 1,000-ft channel dimensions	4.20E+06	-1.20E+06	3.60E+06	2.40E+06
3H*	Straight seaward from off SR-105 dike	2.10E+06	-1.40E+06	5.70E+05	-7.90E+05
4A*	Middle Channel bar dredging	2.20E+06	-3.80E+05	5.30E+05	1.40E+05
4B	Bay Center training structure with 4A				
4C	Dredging across whole entrance with 4B	9.40E+06	-3.90E+05	5.70E+05	1.80E+05
4D	Willapa River training structure with 4A				
4E*	4A, but 38- x 1,000-ft channel dimensions	1.20E+07	-1.60E+06	2.80E+06	1.20E+06
	Grand Average	4.60E+06	-1.20E+06	1.20E+06	-5.90E+04
	Average for 1,000- x 38-ft cross section			2.20E+06	
	Average for 500 –x 28-ft cross section			5.20E+05	
	Average for Template 3B			4.00E+04	

*Alternatives that passed screening.

The concept of a migrating Channel Alternative is to allow the authorized and buoyed channel to be moved slightly from year to year within prescribed bounds to take advantage of the fairly constant channel cross section historically maintained by nature even as the channel migrated. Because of the strong channel migration cycle at Willapa, this means that the maintained channel would shift systematically southward, rotating counterclockwise, and usually increasing in curvature over a series of years. When the natural channel no longer supports safe navigation, a new channel will be dredged through the spit in a position to provide favorable navigation for a number of subsequent years. The number of such channel relocations can be estimated from the geomorphic history of natural channel migration.

Avoidance of extreme curves and south heading. The Port of Willapa Harbor identified the 1999 channel as the maximum safe southward channel deflection. In the past, deflection created unsafe conditions for years in succession (Appendix A). Examples of extreme unsafe deflection are shown in Figure 3-24 along with the worse-case acceptable condition under Alternative 3B. If Alternatives 3B or 3G were adopted, dredging would be necessary at intervals to force establishment of a new northwest outlet along the line of Alternative 3A or 3F, respectively. The number of years nature has taken to complete channel cycles and establish a new northwest outlet are tabulated in Table 3-6. Also shown are the number of years before these new outlets captured the main flow, before unacceptable channel deflection reoccurred, and finally before the next outlet was initiated.

Timing of the natural cycle with navigational requirements. Table 3-6 shows five episodes during which new outlets formed an average of 1.6 years after the channel developed an undesirable deflection (DUD). An average of 4.5 years elapsed after DUD before the new outlet completely dissected the Cape Shoalwater spit to a depth of 18 ft mllw and the severed shoal began migrating away from the spit. An average of 6 years after DUD, the new outlet captured the main flow of the north channel. Historically, the Coast Guard moved marker buoys to the new outlet at about this time (6 years after DUD). So, Alternative 3B and 3G would have to shorten the natural 12-year channel cycle by 6 years on the average to avoid undesirable deflection. This analysis highlights the distinction between the Seattle District's past dredging practice (which followed the migrating channel until the new outlet dominated, Table 3-6) and Alternatives 3B and 3G which would relocate the channel as needed. Again, 1998-99 was not a typical year. The naturally occurring new outlet gained dominance in time so that no relocation would have been required to switch the authorized channel back to the northwest in time to avoid an unsafe curved channel.

The lifetime dredging requirement for a migrating Alternative is the sum of initial dredging, annual maintenance, and occasional relocation dredging. A modest volume would be dredged annually to eliminate isolated shoals and facilitate continuous marking of the channel. Larger volumes will be dredged less frequently to avoid unsafe navigation by relocating the entire approach channel. To apply this concept, the project lifetime is set to 50 years, the duration of low-maintenance dredging is set to 6 years, and typical shoaling rates are inferred from rates of spit growth and shoaling within the templates (Tables 3-3 and 3-4).

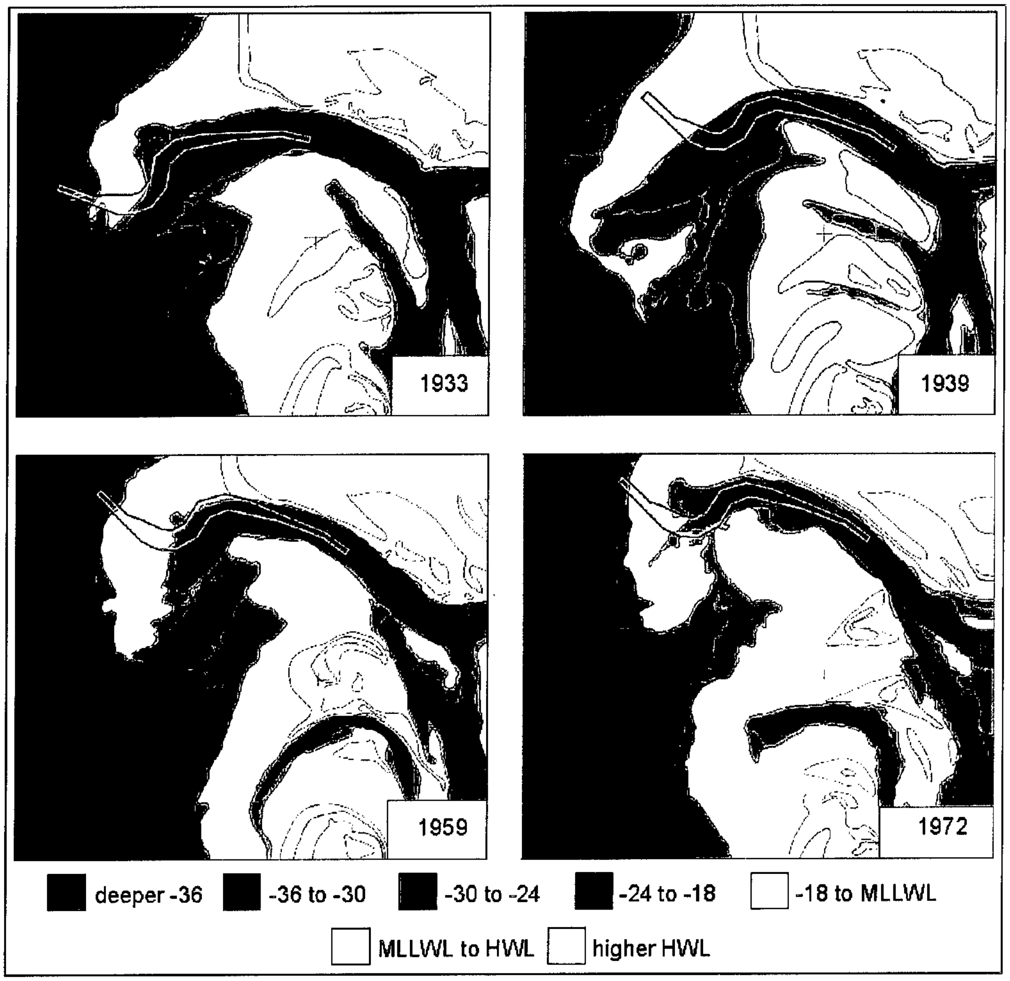


Figure 3-24. Alternative 3B avoids tight curve and long north-south run typical of natural channels

The resulting dredging estimates are thus based on simplifying assumptions, past natural bathymetric changes, and an average channel cycle description assuming the past can serve as an indication of the future. Furthermore it is assumed the new dredging project will not significantly alter the long-term rates or cycles. The only known mechanism that would change past rates is not quantifiable, but will be discussed in the last section of this chapter.

Combining components into a lifetime estimate. For Alternatives 3G and 4E a typical volume change rate of 2.5 million cu yd/year is taken as representative of the measurements combined from different approaches (Figure 3-16, Table 3-3 and 3-4). This background shoaling rate is multiplied by 50 years to estimate the lifetime estimate for these two Alternatives. For migrating Channel Alternative 3B, it is assumed that a new outlet will have to be dredged across the Cape Shoalwater spit at intervals of every 6 years. After calculating the volume that Template 3B would cut into bathymetry from different years, a typical value of 1 million cu yd was adopted to represent the

Table 3-6
Basis for Mixture of Low- and High-Volume Dredging Requirements for Templates 3B and 3G

Events in the Willapa Channel Cycle					Years Between Undesirable Navigation and New Outlet			
Initiation of New Outlet	**Dissection of Shoal**	**New Outlet Dominates**	**Navigation Switches**	**Undesirable Curve for Navigation**	**Initiation**	**Dissection**	**Dominance**	**Navigation Switched**
1886								
1918	1922	1924	1924	1932				
1933	1934	1936	1935	1939	0	2	4	3
1940	1942	1945	1945	1953	1	3	6	6
1954	Outlet initiated in 1954 grows and migrates south for 5 years, but fails to capture the main flow and shoals over in 1959.							
1959	1960	1961	1961	1967	6	7	8	8
1970	1973	1973	1973	1999	3	6	6	6
1997	200?	200?+	1999		-2			
				Average Life	1.6	4.5	6	5.75

Column 1 gives the year of the chart first showing evidence of a new outlet starting to erode into the Cape Shoalwater Spit.
Column 2 shows the years that new outlets extended entirely across the ebb shoal with depths >18 ft.
Column 3 shows the years that new outlets became larger in cross section than the older outlet.
Column 4 indicates the first year when District records indicate navigation switched to the newer outlet.
Column 5 indicates the first year when the navigation channel developed an S-shape and the North-to-South passage that appears similar to the 1998 condition was identified by the Port of Willapa Harbor as barely acceptable for navigation.

larger dredging amount that would relocate the channel every 6 years. The amount of dredging that would have been required for the initial cut of Alternative 3B into 1998 was similar to the amount that shoaled into 3B during the 1998-99 period (Table 3-3). This average was adopted as the lower maintenance estimate for the years between major channel relocations. Estimates for the other four Alternatives were derived using the relationship between large and small template trapping shown in Figure 3-18. Assumptions and final estimates are summarized in Table 3-7.

This concludes a description and application of a methodology that should be reapplied with revised input (especially for the initial dredging requirements) if the project is not adopted until some later phase of the channel cycle.

Discussion of Geomorphic Findings

A prominent channel occupied the North Fairway on all of the examined historical charts (1952-1984) and all of the recent surveys. Channels first appeared in the Middle Fairway on the 1928 chart and in the South Fairway in 1936. Middle and South fairways had clear channels on less than 20 percent of the full-entrance charts produced between 1852 and 1978. This stark difference in frequencies of channels in the three fairways suggests that conditions only infrequently favored a short-lived Middle or South Fairway channel. There is no indication that this tendency changed in recent years.

Table 3-7
Lifetime Dredging Estimates

Components from Geomorphic Analysis	
2.27 to 3.70	Background shoaling rate (million cu yd/year) measured in large cross-section at fixed locations which agrees with independent results from the spit-growth approach (Figure 3-16)
0.45 to 0.81	Background shoaling rate (M cu yd/year) for Alternative 3B
1.00 to 1.67	Estimated dredging required to relocate 3B (M cu yd)
Y = 2.21X+.44	Relationship between large (Y) and small (X) cross-section trapping (M cu yd/year)
1.72	Standard deviation of estimated annual shoaling rate within channel templates (ft)
6	Interval between required relocate of Alternatives 3G and 3B (years)
50	Project life (years)

Estimated 50-year Dredging Requirement, million cu yd/year

	Excluding Initial Dredging		Total With Initial Dredging Based on Conditions Like Those of 1998 (Table 3-5)	
Template	Lower Estimate	Upper Estimate	Lower Estimate	Upper Estimate
3A	42	74	43	75
3B	27	48	27	48
3F	114	185	120	191
3G	58	102	62	106
3H-a	42	74	44	76
3H-b	42	74	44	76
4A	42	74	44	76
4E	114	185	126	197

There is no significant trend to bar depths with time for any fairway. Bar depth and orientation of the channel over the bar follow patterns consistent with a cycle of approximately 12-year average duration for the North Fairway. Undesirable alignment of the bar channel with respect to the waves has been a major obstacle to safe passage in and out of Willapa Bay (Chapter 2, Chapter 3). Changes in orientation of smaller channels in the South and Middle are linked to the same 12-year cycle that dominates the North Fairway. South and Middle fairway channel size is, however, more closely related to a much longer term trend. A minor channel began off Leadbetter Point in 1945, grew gradually, and migrated slowly north to finally become the prominent channel in the middle of the entrance on the 1993 63rd edition National Oceanic and Atmospheric Administration chart of Willapa Bay. This channel represented the culmination of a 50-year growth-migration trend that ended around 1992. As evidenced in the 1998 and 1999 surveys, the middle channel continues diminishing as the ocean end fills.

The initial dredging quantities that would have been required in 1998 were largest for Middle Channel Alternative 4A and smallest for North Channel Alternative 3B. Rapid changes occurred between the 1998 and 1999 surveys, so that Alternative 3A and 3B share the designation as the alternatives requiring least initial dredging for 1999.

Two independent approaches for estimating maintenance requirements indicated shoaling rates of at least several million cubic yards/year for any fixed-position channel located in the northern two-thirds of the entrance (North or Middle Fairways). This estimate pertains to typical years with potentially many more millions of cubic yards fill during severe years. Although the geomorphically based shoaling rates depend on assumptions considered as only approximately valid, internal consistency suggests these large inferred rates are reasonable, but probably low. The assumption used to derive these rates, that the volume of sand bypassing channel templates during surveyed years is the same as would bypass the channel Alternatives, underestimates shoaling by an amount that can not be quantified from geomorphology. Two observations suggest underestimation. First, without increased flushing, bigger channels will tend to trap more sediment per unit channel area than natural channels. Second, depth and width parameters tabulated from the historical charts were not significantly greater for years when charts were based on postdredging soundings than they were based on either predredging charts or charts depicting conditions when no dredging had occurred for a year or more. This lack of identifiable increases in channel size attributable to dredging suggests that rapid shoaling can quickly fill dredged (artificially deepened) areas returning the total cross-sectional area to equilibrium. Historical conditions never included deeply dredged channels that correspond to the proposed Alternatives. So, past bathymetric changes do not include direct evidence of the increased shoaling expected with the larger proposed Alternatives. A portion of this increased shoaling rate related to channel size was inferred empirically (Figure 3-18), but results in this chapter do not include any increase corresponding to increased specific trapping efficiency.

Combining initial and maintenance estimates from the geomorphic analyses indicates the migrating Alternatives, 3B and 3G, would minimize lifetime dredging requirements. In contrast, Alternatives 3F and 4E would require the most dredging. Dredging estimates are intermediate for the other Alternatives. Further distinction among Alternatives on the basis of lifetime of dredging requirements inferred strictly from historic changes does not appear warranted. The infrequent occurrence of any middle channel does, however, compound concern that Alternatives 4A and 4E would shoal rapidly.

The long-term steadiness of natural channel dimensions (tabulated over the bar), the small difference between the shallower design depth and the apparent equilibrium bar depth in the North Fairway, and the low rate of bathymetric change calculated by repositioning channel templates from year to year (to follow the natural channel) show that considerable savings are possible if the authorized channel were allowed to migrate. The maintained channel did migrate during earlier dredging episodes. Future plans would avoid past unsafe north-to-south deflection of the channel by relocating the channel to the north about 6 years earlier than nature normally moves the main flow. Artificially advancing the natural timetable of channel relocation was not part of the earlier dredging programs.

Allowing the channel to follow nature within safe limits is the geomorphically based key for reducing dredging requirements. Other channel design criteria must, however, be considered in the final evaluation (Chapter 8). In conclusion on geomorphology, two migrating Channel Alternatives were postulated at the onset of the study. The other Alternatives were stationary by

definition. Some of the cost savings achievable with migrating channels could be incorporated into the other North Channel Alternatives by modification of definitions given for this study.

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4 Field Data Collection and Analysis[1]

The Willapa Bay field data collection program was executed by Evans-Hamilton, Inc. (EHI) under task-order contract with the U.S. Army Engineer Research and Development Center (ERDC), Vicksburg, MS. The program incorporated measurements of waves, currents (point measurements and profiles through the water column), water level, salinity (conductivity and temperature), wind velocity, air temperature, and air pressure. Each of three field deployments included five wave and current stations, four combined water-level and salinity recording stations, and one weather station. Wave and current instruments were arranged on the same bottom mounts, and stations were divided geographically into two groups as the entrance gauges and as the inner, or back-bay, gauges. Water-level recording stations were distributed around the periphery of the bay and in the river channels. The weather (meteorological) station was deployed near the center of Willapa Bay, approximately 10 m above the water surface. The instruments stored data internally, with measurement times referenced to Greenwich Mean Time (GMT). Two cruises were conducted to measure baywide variations in the current structure and to define the vertical density (salinity) structure of the water column.

Data collection was conducted both for project-specific objectives and for documenting the existing physical processes as a baseline for monitoring changes, should channel dredging take place. Project-specific objectives concerned primarily acquisition of data to verify numerical simulation models of the bay hydrodynamics (Chapter 6) and the waves at the entrance to Willapa Bay (Chapter 5). The data also served to characterize the hydrodynamic environment at the Willapa Bay entrance for assessing navigation reliability of the various project design alternatives discussed in Chapter 2.

The data collection program was initially developed at the ERDC Coastal and Hydraulics Laboratory (CHL), to meet the project objectives, including selection of instruments, rates and duration of sampling, and length of deployments. EHI subsequently reviewed and offered recommendations for the program. The instrument types and suites were based on experience of CHL in data collection at the mouth of the Columbia River and in other projects. EHI

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constructed mounts to house the instruments based upon cumulative experience in data collection at Willapa Bay and similar ocean-estuary environments.

Wave Measurements

A combination of directional and standard wave measurements (wave height and period) was made during the Willapa Bay field effort. Two types of systems were deployed, and station locations are shown in Figure 4-1. For the entrance gauges (Stations 1, 2, 3 and 10, 11, 12), directional wave measurements were made with SonTek, Inc., Hydra instruments incorporating a SonTek Acoustic-Doppler Velocimeter (ADVOcean) probe coupled with a Paroscientific, Inc., digi-quartz pressure sensor. Current and pressure readings were collected once an hour, starting at the top of the hour, and instruments were set to collect 4,096 data points at 2 Hz. For the back-bay gauges (Stations 4, 5, and 13), pressure readings were collected with a digi-quartz pressure sensor connected externally to a SonTek Acoustic Doppler Profiler (ADP). Pressure readings were collected once an hour, starting 1.5 min before the hour, recording 1,024 data points at 1 Hz.

The ADPs coupled with a Paroscientific pressure sensor operated with two sampling schemes, one for measuring currents and one for measuring waves. Because the current data were averaged over 3-min intervals and centered at the top of the hour, the pressure data had to be synchronized with the current readings, causing the awkward start time. All pressure sensors had a measuring range of 0-45 psia. Field service dates and the parameters measured at each station are summarized in Table 4-1.

Instruments were deployed on the seabed on two types of mounts, either on a tripod or on a trawl-resistant frame. Mounts were selected based on the anticipated sediment type, predicted waves and currents, and site-specific characteristics that would have bearing upon the success of the data collection effort. Tripods provide the flexibility of mounting multiple instruments on one deployment package while maintaining a stable platform. They also provide some protection from instrument burial by shifting sand. Figure 4-2 shows a tripod with an ADP current meter and a Hydra system mounted on the tripod.

The trawl-resistant frames were originally designed to provide protection for the instruments mounted in the frame and to incorporate a low profile to avoid damage by local vessel activity. Trawl-resistant frames also possess a wider footprint to hinder settling in softer sediments.

Before deployment, one or two cross-channel bathymetry transects were obtained to aid the placement of each instrument package. Deployment and recovery positions were recorded during both deployment and retrieval cruises. The survey vessel was equipped with differential Global Positioning System (GPS) (Trimble DSM-Pro), which provides ±1-m accuracy. Water depths were measured with an echosounder on the survey vessel. Bar checks were conducted to calibrate the echosounder. Deployment and recovery information is summarized in Tables 4-2 and 4-3.

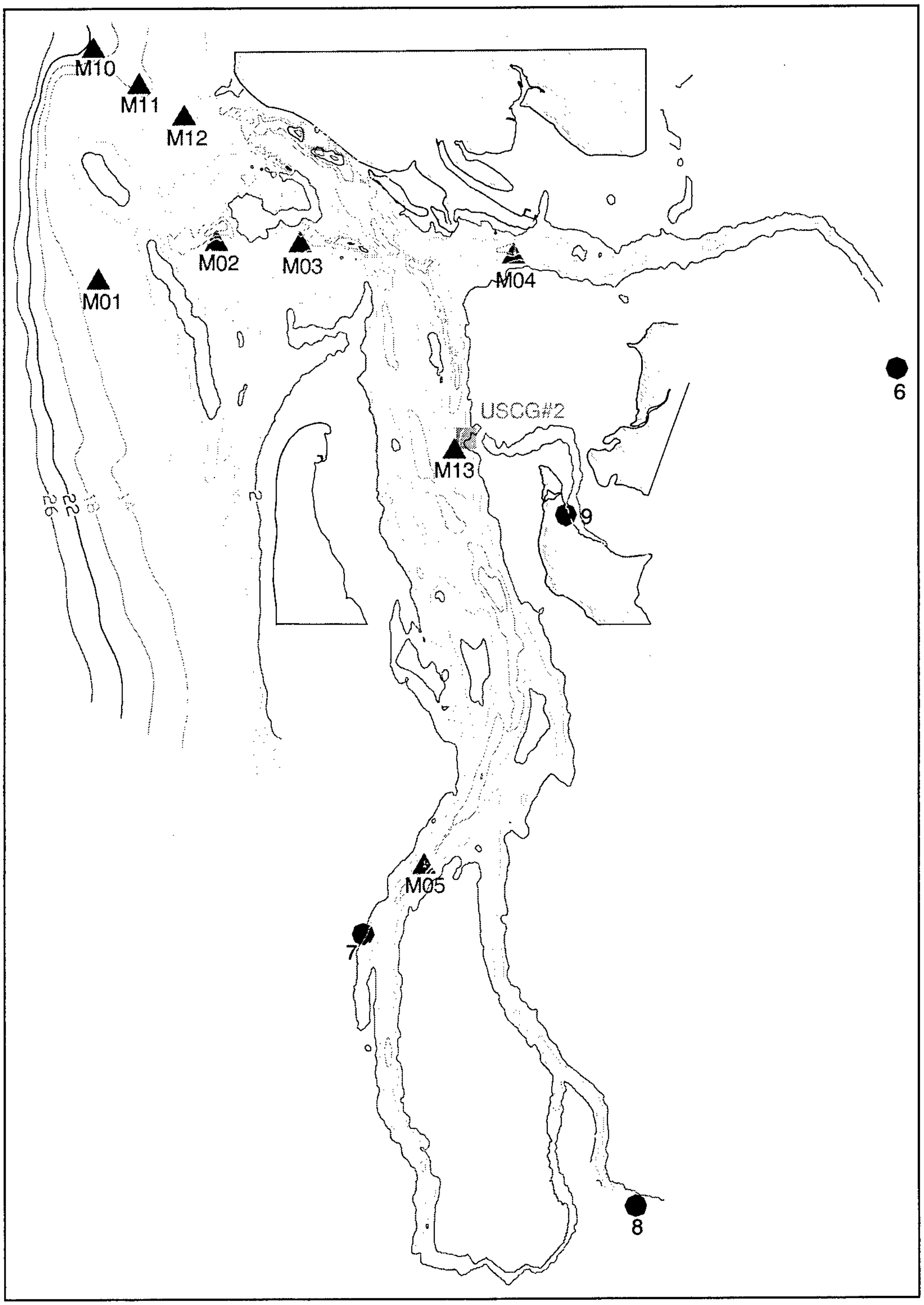

Figure 4-1. Location of current meters and wave gauges (triangles), water-level gauges (circles), and meteorological station (squares) in Willapa Bay

Table 4-1
Willapa Bay Data-Collection Schedule

Station No.	Parameters Measured	1998 Deployment 1 August	Deployment 2 September	Deployment 3 October	Deployment 4 November	December
Waves						
1[1]	Current speed, current direction, temperature, pressure, pitch, roll, and heading	26th	-10th 10th	-9th 9th	-18th	
2[1]		25th	-9th 11th	-7th 14th	-17th	
3[1]		25th	-9th 11th	-7th 14th	-11th	
10[1]						
11[1]						
12[1]						
Wave Height & Period						
3	Pressure				17th-	16th
4		26th	-9th 10th	-13th 14th	-11th	
5		25th	-10th 10th	-12th 14th	-12th	
13					17th-	17th
Currents						
1	Current speed, current direction, temperature, pitch, roll, and heading	26th	-10th 10th	-9th 9th	-18th	
2		25th	-9th 11th	-7th 14th	-17th	
3		25th	-9th 11th	-7th 14th	-11th 17th-	16th
4		26th	-9th 10th	-13th 14th	-11th	
5		25th	-10th 10th	-12th 14th	-12th	
13					17th-	17th
MicroCAT						
1	Conductivity and temperature	26th	-10th 10th	-9th 9th	-18th	
Water- Level Gauges						
4	Conductivity, temperature, and pressure	26th	-9th 10th	-13th 14th	-11th	
6 (South Bend)		13th	-16th 16th	-21st 21st	-17th 17th-	17th
7 (Nahcotta)		12th	-15th 15th	-22nd 22nd	-17th 17th-	18th
8 (Naselle River)		12th	-15th 15th	-22nd 22nd	-18th 18th-	18th
9 (Bay Center)		11th	-16th 16th	-21st 21st	-17th 17th-	17th
13					17th-	17th
Meteorological Station						
Bay Center Channel Light, Number 2	Wind speed, wind direction, air temperature, and barometric pressure	12th	-11th 11th	-7th 7th	-17th 17th-	17th
Field Service Dates						
Waves/Currents		24th - 26th	9th-11th	7th-9th & 11th-14th	11th-13th & 16th-18th	16th - 17th
Water-Level Gauges (Stations 6-9)		11th - 13th	15th - 16th	21st - 22nd	17th - 18th	17th - 18th
Meteorological Station		12th	4th & 11th	7th	17th	17th
ADCP Transects[2]					17th - 18th	
Profiling CTD Survey[3]					16th – 18th	

[1]Directional.
[2]ADCP = Accoustic Doppler Current Profiler.
[3]CTD = Conductivity, temperature, and depth (pressure) sensor – primarily deployed to measure water salinity as a function of depth.

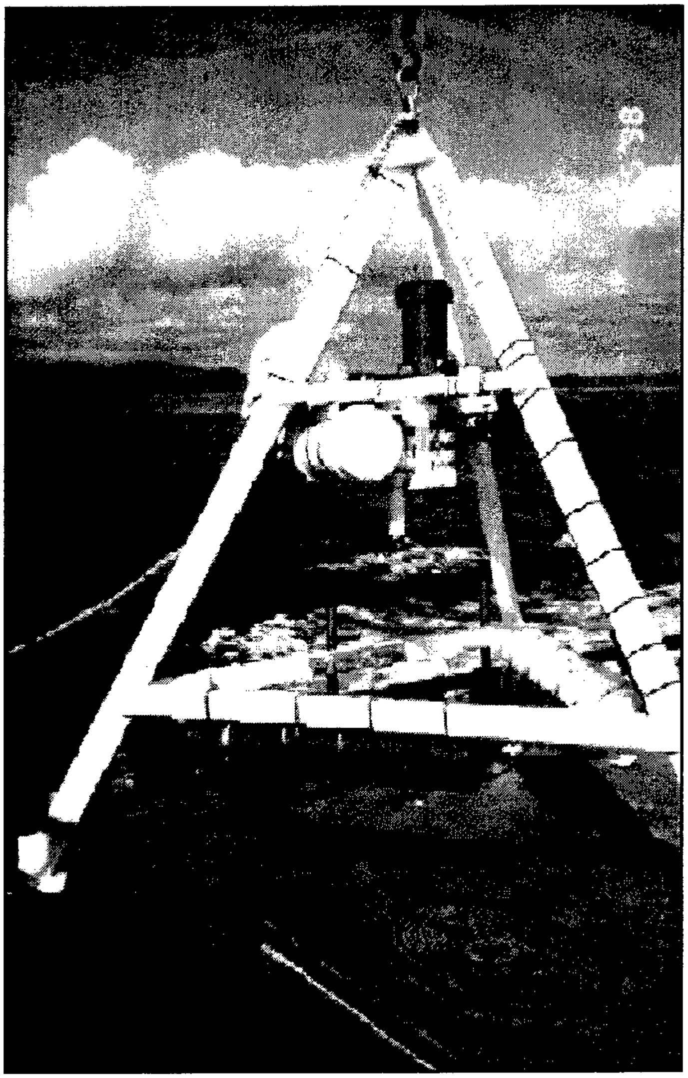

Figure 4-2. Tripod mount as deployed in Willapa Bay

Data reduction and quality control procedures for wave measurements

Processing and analysis of the Willapa Bay wave data were performed by Pacific International Engineering (PIE) under the direction of ERDC. The work included quality assurance checks of the recorded data and spectral analysis. Five stations were occupied; the three located at the entrance measured directional waves, and the two in the bay measured non-directional waves.

Wave data from each deployment were first checked for inconsistencies by developing time-series plots of heading, pitch, and roll recorded by each ADVOcean or ADP instrument. Pressure time-series were generated and analyzed to identify discrepancies in the record. Deployment and recovery information is summarized in Table 4-2.

Table 4-2
Tripod Deployment and Retrieval Locations

Station No.	Date GMT	Position Latitude (N) Longitude (W)	Position[1]	Time GMT	Water Depth[2]	Date GMT	Position Latitude (N) Longitude (W)	Position[1]	Time GMT	Water Depth[2]	Comments
Deployment No. 1						**Retrieval No. 1**					
1	08/26/98	46 41.002' N 124 08.751' W	X 725,726 Y 513,536	18:00	38 (11.6 m)	09/10/98	46 41.001' N 124 08.745' W	X 725,750 Y 513,492	17:59		
2	08/25/98	46 41.663' N 124 05.843' W	X 738,057 Y 516,908	23:21	33 (10.1 m)	09/09/98	46 41.664' N 124 05.848' W	X 738,037 Y 516,917	18:14		ADP missing profiles because of hardware problems.
3	08/25/98	46 41.818' N 124 03.817' W	X 746,563 Y 517,464	23:02	33 (10.1 m)	09/09/98	46 41.818' N 124 03.817' W	X 746,563 Y 517,464	18:30		ADVOcean flooded, disabling compass, tripod hard to pull up.
4	08/26/98	46 41.756' N 123 58.371' W	X 769,299 Y 516,072	16:38	32 (9.8 m)	09/09/98	46 41.755' N 123 58.387' W	X 769,230 Y 516,073	18:58		Tripod spun ~125 deg after ~4 days, depth & position the same.
5	08/25/98	46 31.286' N 123 59.956' W	X 759,855 Y 452,796	18:07	40 (12.2 m)	09/10/98	46 31.268' N 123 59.830' W	X 760,379 Y 452,661	21:34	18 (5.5 m)	Mount flipped after ~2 days and moved ~500 ft east.
Deployment No. 2						**Retrieval No. 2**					
1	09/10/98	46 41.023' N 124 08.717' W	X 725,871 Y 513,576	18:30	37 (11.3 m)	10/09/98	46 41.002' N 124 08.710' W	X 725,896 Y 513,533	18:59		ADVOcean leaked, disabling compass, ADP missing profiles.
2	09/11/98	46 41.765' N 124 05.834' W	X 738,122 Y 517,524	18:22	40 (12.2 m)	10/07/98	46 41.763' N 124 05.834' W	X 738,120 Y 517,516	15:22		
3	09/11/98	46 41.800' N 124 03.747' W	X 746,849 Y 517,343	18:49	33 (10.1 m)	10/07/98	46 41.801' N 124 03.754' W	X 746,820 Y 517,352	15:49		Large green seaweed caught on tripod, hard to pull up.
4	09/10/98	46 41.757' N 123 58.423' W	X 769,081 Y 516,091	16:35	33 (10.1 m)	10/13/98	46 41.757' N 123 58.423' W	X 769,081 Y 516,091	00:25		Tripod sunk ~4 ft, large green seaweed caught on tripod.
5	09/10/98	46 31.296' N 123 59.995' W	X 759,692 Y 452,859	23:29	46 (14.0 m)	10/12/98	46 31.300' N 123 59.990' W	X 759,715 Y 452,881	22:35		Mount appears to be acting like a sediment trap.
Deployment No. 3						**Retrieval No. 3**					
1	10/09/98	46 41.005' N 124 08.721' W	X 725,859 Y 513,732	19:29	39 (11.9 m)	11/18/98	46 41.043' N 124 08.720' W	X 725,865 Y 513,698	18:00	40 (12.2 m)	
2	10/14/98	46 41.850' N 124 05.841' W	X 738,110 Y 517,919	21:54	30 (9.1 m)	11/17/98	46 41.849' N 124 05.897' W	X 737,883 Y 518,047	00:04	21 (6.4 m)	Leg broke off tripod during retrieval.
3	10/14/98	46 41.851' N 124 03.848' W	X 746,444 Y 517,675	22:11	33 (10.1 m)	11/11/98	46 41.853' N 123 03.842' W	X 746,467 Y 517,681	20:28	27 (8.2 m)	
4	10/14/98	46 41.749' N 123 58.487' W	X 768,814 Y 516,054	18:42	31 (9.4 m)	11/11/98	46 41.759' N 123 58.479' W	X 768,849 Y 516,113	21:09	22 (6.7 m)	
5	10/14/98	46 31.319' N 123 59.920' W	X 760,017 Y 452,987	20:17	39 (11.9 m)	11/12/98	46 31.338' N 123 59.938' W	X 759,943 Y 453,105	23:20	40 (12.2 m)	
Deployment No. 4						**Retrieval No. 4**					
3	11/17/98	46 41.812' N 124 03.824' W	X 746,534 Y 517,432	00:26	22 (6.7 m)	12/16/98	46 41.812' N 124 03.827' W	X 746,516 Y 517,428	18:46	32 (9.8 m)	
13	11/17/98	46 38.532' N 123 59.710' W	X 762,838 Y 496,743	22:46	24 (7.3 m)	12/17/98	46 38.556' N 123 59.709' W	X 762,844 Y 496,891	13:00	30 (9.1m)	

[1] In feet referred to the North American Datum of 1983.
[2] In feet referred to mean lower low water (mllw).

Table 4-3
Willapa Bay Equipment Log

Deployment					Acoustic Doppler Profiler (ADP)					ADVOcean			PAROS		MicroCAT	
Deployment Number	Station Number	Water Depth[1]	Deployment Date	Retrieval Date	Serial Number	Frequency kHz	Distance of Transducer off Bottom m	Cell Size m	Blank Dis. M	Serial No.	Frequency MHz	Distance of Transducer off Bottom m	Serial No.	Distance to Pressure Port off Bottom m	Serial No.	Distance off Bottom m
1	1	11.6	08/26/98	09/10/98	C50	1,500	1.93	0.5	0.4	B62	5.0	1.09	70621	1.28	37SI19004-0687	1.86
	2	10.1	08/25/98	09/09/98	C35	1,500	1.93	0.5	0.4	B94	5.0	1.09	72688	1.28		
	3	10.1	08/25/98	09/09/98	C36	1,500	1.93	0.5	0.4	B107	5.0	1.09	72690	1.28		
	4	9.8	08/26/98	09/09/98	C30	1,500	1.93	0.5	0.4				72689	1.28		
	5	12.2	08/25/98	09/10/98	C32	1,500	0.47						70361	0.51		
2	1	11.3	09/10/98	10/09/98	C35	1,500	1.93	0.5	0.4	B94	5.0	1.09	72688	1.28	37SI19004-0687	1.86
	2	12.2	09/11/98	10/07/98	C50	1,500	1.93	0.5	0.4	B62	5.0	1.09	70621	1.28		
	3	10.1	09/11/98	10/07/98	C36	1,500	1.93	0.5	0.4	B73	5.0	1.09	72690	1.28		
	4	10.1	09/10/98	10/13/98	C30	1,500	1.93	0.5	0.4				72689	1.28		
	5	14.0	09/10/98	10/12/98	C32	1,500	0.47	0.5	0.4				70361	0.51		
3	1	11.9	10/09/98	11/18/98	C50	1,500	1.93	0.5	0.4	B62	5.0	1.09	70621	1.28	37SI19004-0687	1.86
	2	9.1	10/14/98	11/17/98	C35	1,500	1.93	0.5	0.4	B118	5.0	1.09	72688	1.28		
	3	10.1	10/14/98	11/11/98	C36	1,500	1.93	0.5	0.4	B73	5.0	1.09	72690	1.28		
	4	9.4	10/14/98	11/11/98	C30	1,500	1.93	0.5	0.4				72689	1.28		
	5	11.9	10/14/98	11/12/98	C32	1,500	0.47	0.5	0.4				70361	0.51		
4	3	6.7	11/17/98	12/16/98	C30	1,500	1.93	0.5	0.4				72689	1.28		
	13	7.3	11/17/98	12/17/98	C32	1,500	0.47	0.5	0.4				70361	0.51		

[1]In meters relative to mean lower low water.

Equipment locations, sampling configurations, and mounting parameters are summarized in Table 4-3. As noted in Table 4-2, one ADVOcean flooded during two separate deployments, shorting out the compass card. The compass card measures pitch, roll, and heading, which are used to convert the three measured beam velocities to velocities referenced to earth coordinates. Current speeds from the flooded ADVOcean looked reasonable, but the directions were obviously wrong. Data were corrected by using the compass values from the ADP mounted on the tripod above the ADVOcean, and incorporating the ADP compass values to calculate velocities referenced to earth coordinates. Quality checks for all deployments determined that the data were good.

Wave energy spectral analysis was performed to determine adequate high-frequency cutoff values. Energy distributions by frequency and direction were plotted for each data burst collected at each station. Typical spectra for the offshore station showed a single or double peak arriving at periods between 8 and 16 sec, indicating swell arriving from offshore (Figure 4-3).

Energy spectra for stations located east of the outer bar (inside the mouth of the bay) did not usually exhibit a distinct energy peak, with energy spread across frequency (periods between 4 and 10 sec). Figure 4-4 shows typical spectra at Station 2 (inside bay) throughout a tidal cycle. The plots show a decrease in energy density as ebb current speed increases and an increase in energy density as the flood current speed increases.

The differences in spectral shape between the offshore station and those inside the bay (specifically changes in wave energy distribution by frequency) are most likely caused by a combination of depth-limited wave breaking over the outer bar and the neglect, in the analysis, of changes in wavelength induced by currents. These changes in wave energy distribution by frequency made the task of choosing a high-frequency cutoff for the inner bay stations difficult. The following high-frequency cutoff recommendations by CHL were applied in the processing:

a. Station 1: 0.25 Hz.

b. Station 2: 0.21 Hz.

c. Station 3: 0.21 Hz.

d. Station 4: 0.25 Hz.

e. Station 5: 0.25 Hz.

Wave data for all stations were analyzed with the wave data analysis standard, spectral analysis program developed by ERDC and the Field Wave Gaging Work Unit at ERDC, Vicksburg, MS.[1] Routine processing of the wave data did not include changes in wavelength produced by wave-current interaction.

[1] Earl, M. D., McGehee, D. D., and Tubman, M. W. (1995). "Field Wave Gaging Program wave data analysis standard," Instruction Report CERC-95-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

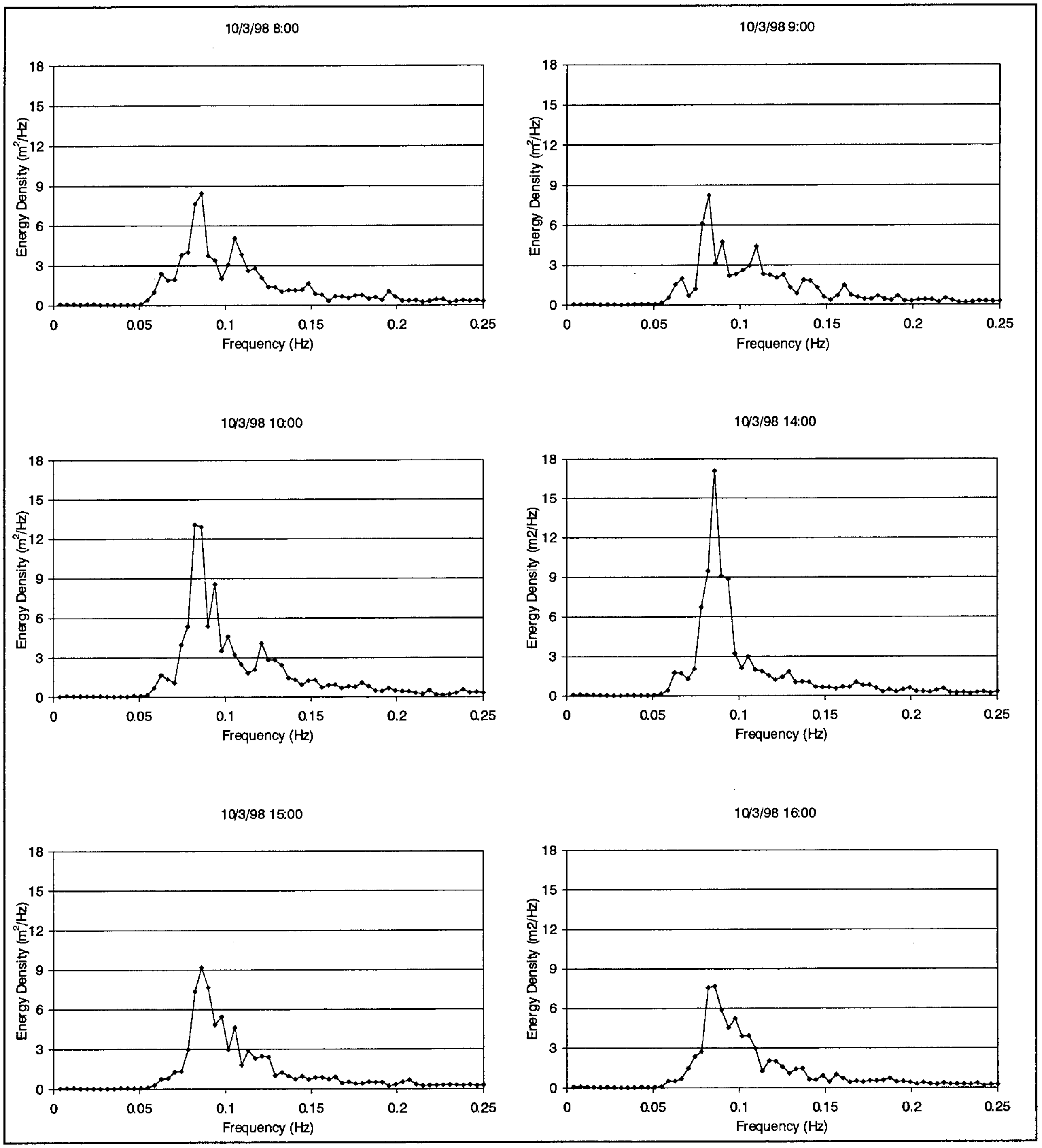

Figure 4-3. Energy density plots for Station 1

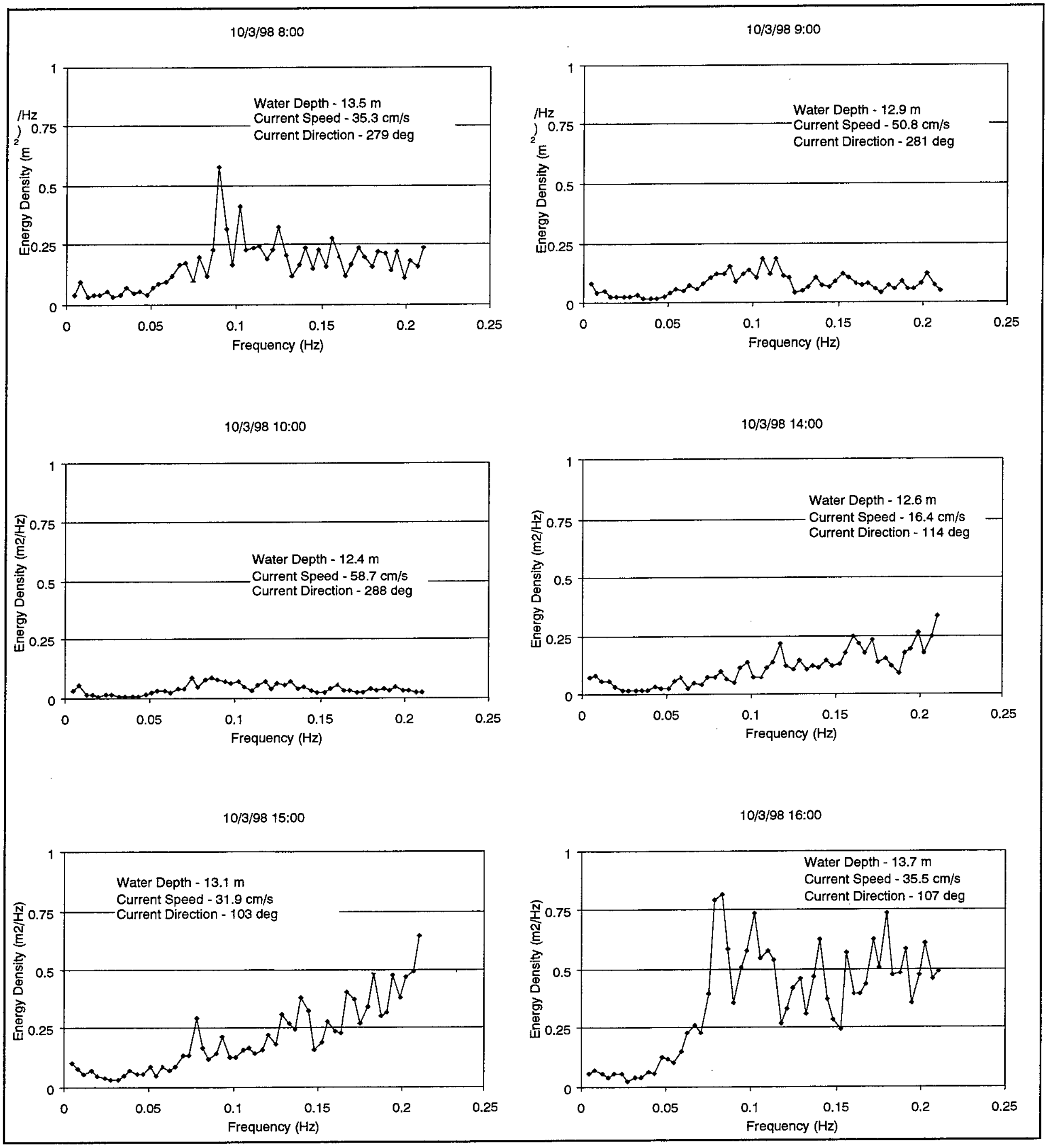

Figure 4-4. Energy density plots for Station 2

Wave height time series were generated and comparisons were made between Stations 1 through 5 and records from the buoy deployed at Grays Harbor, Washington. Figure 4-5 shows the wave height time-history for Deployment 2. Wave heights from Station 1 were similar to the wave heights collected from the Grays Harbor buoy. However, inside the bay, daily changes in wave height and peak period corresponded primarily to changes in water-surface elevation.

Figure 4-6 shows a comparison of Station 1 (west of the outer bar) and Station 2 (east of the bar) wave height and peak period during a typical tidal cycle. Figure 4-7 also shows water depth, current speed, and significant wave height on 3 October 1998.

ADVOcean averages of current speed and direction are shown in Table 4-4 for Deployment 3.

Table 4-4
ADV Data Averages 10-9-98 to 11-18-98

Station No.	Mean Current Speeds cm/sec		Mean Current Direction deg Relative to True North	
	Ebb	Flood	Ebb	Flood
1	16.5	10.9	287.7	76.6
2	41.1	27.3	312.0	111.0
3	44.6	40.5	263.5	99.1

Current Measurements

To define the current regime within the Willapa Bay study area, two measurement strategies were incorporated into the field plan. First, current meters were moored to obtain time series at specific locations (Figure 4-1). Second, to define regional variations in the structure of the current, and to better define the net movement of water in particular channels, the current was measured along transects. Transect data were recorded with a profiling Doppler current meter (RD Instruments (RDI), Acoustic Doppler Current Profiler (ADCP)) mounted over the side of a moving vessel. The instrument is capable of bottom tracking, which removes the velocity of the moving vessel from the recorded current data to retrieve the current velocity profile. Field service dates and the parameters measured at each station are summarized in Table 4-1.

Moored current measurements

Moored current measurements were made with ADPs. Current and wave instruments were deployed on the same bottom mounts as discussed in the wave measurement section (Figure 4-2).

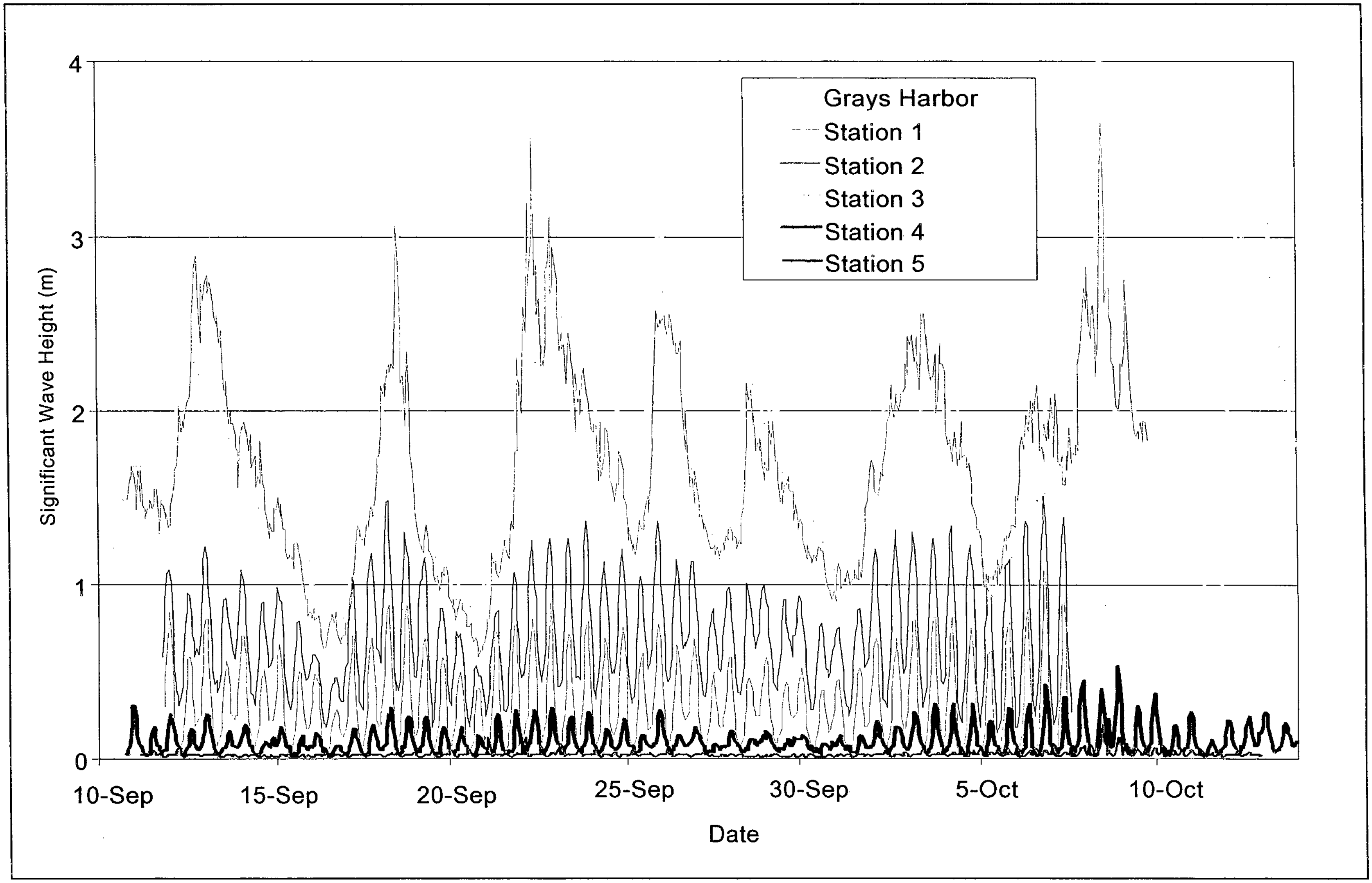

Figure 4-5. Significant wave height time series for Stations 1, 2, 3, 4, 5, and Grays Harbor buoy

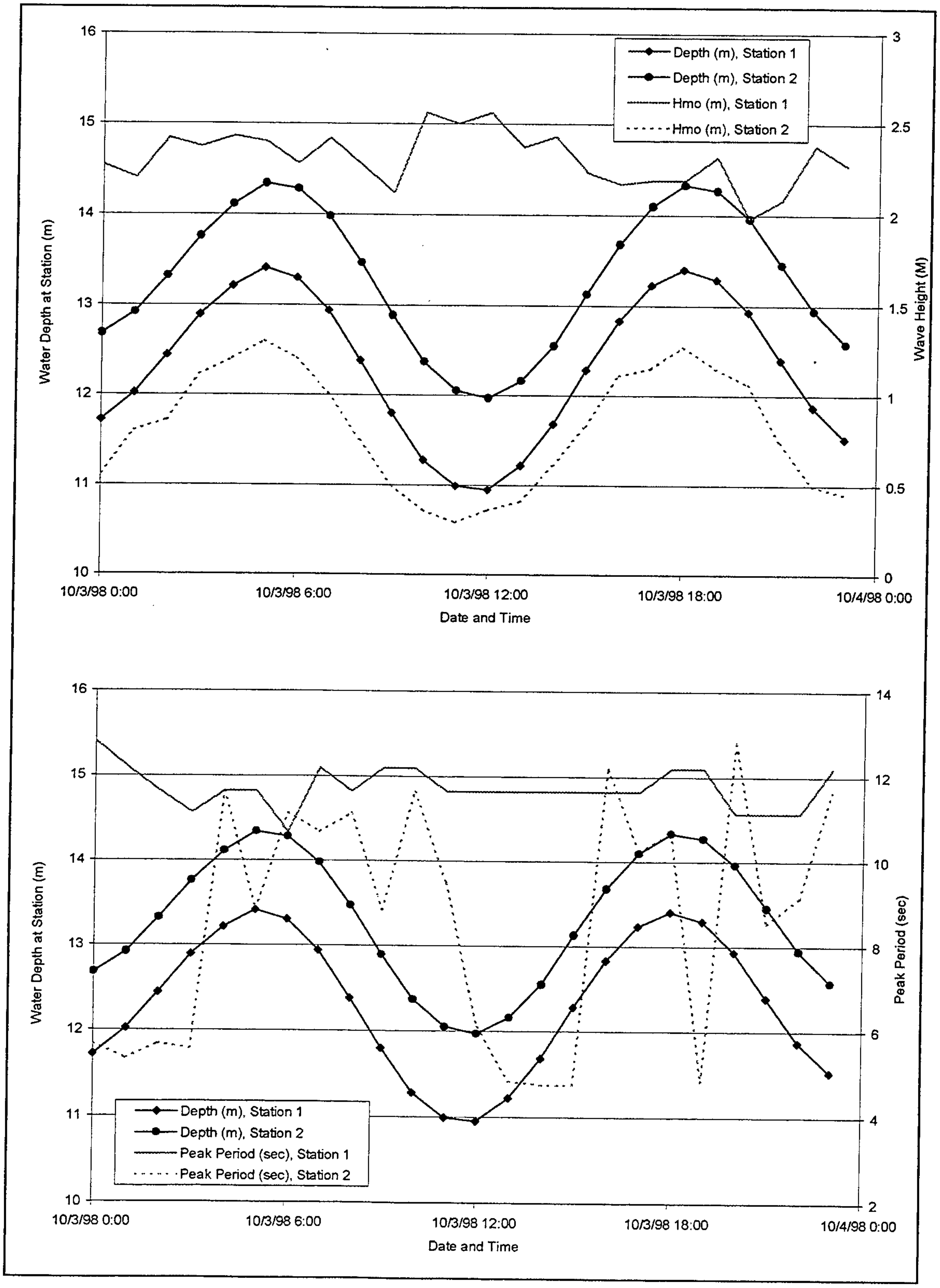

Figure 4-6. Comparison of Stations 1 and 2 wave heights and peak periods

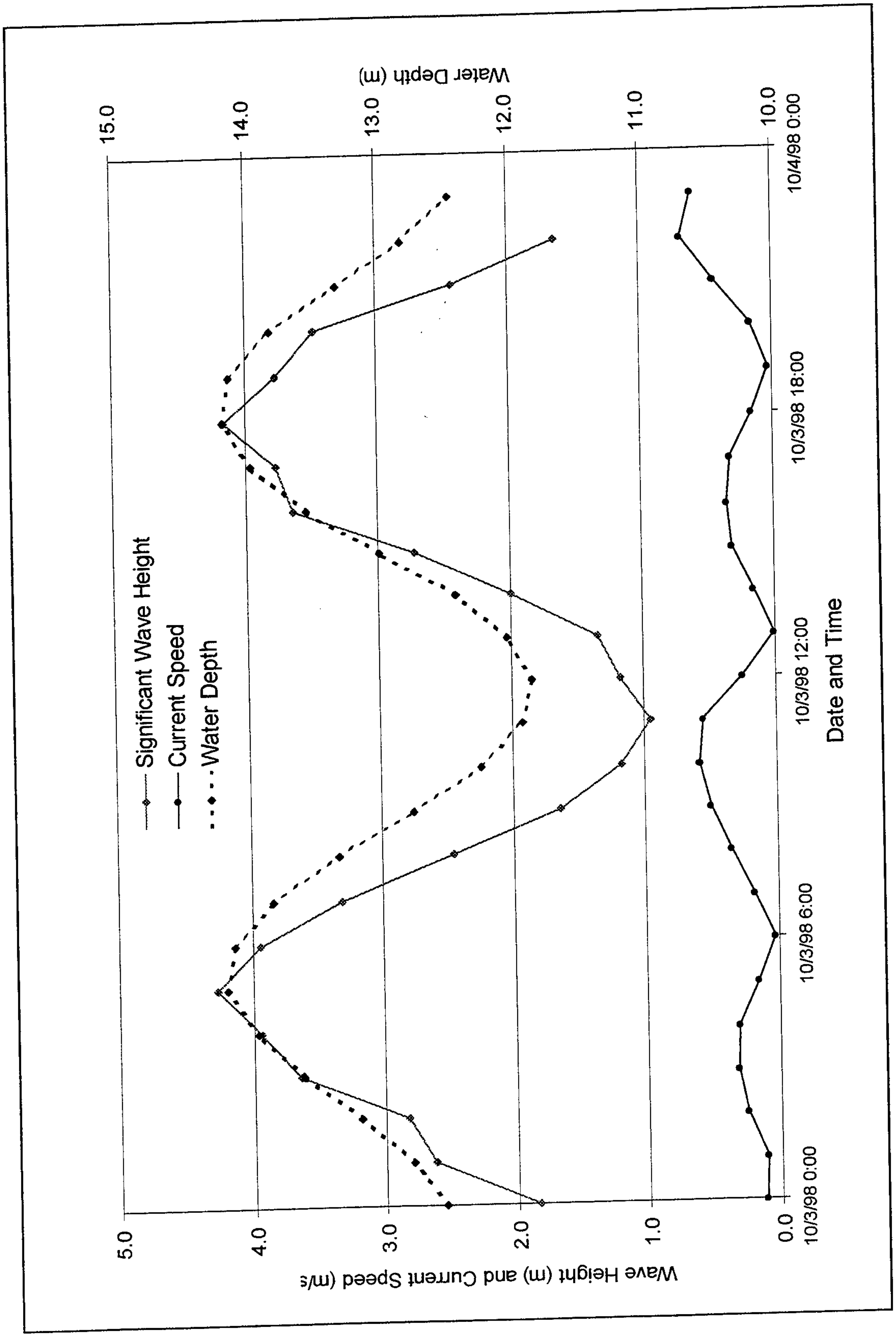

Figure 4-7. Station 2 wave height, water depth, and current speed for 3 October 1998

Five ADPs were deployed. Three of the five ADPs were configured with optional external sensors. One ADP was configured with a Sea-Bird Electronics, Inc., MicroCAT, and two ADPs were configured with Paroscientific, Inc., digi-quartz pressure sensors. Equipment locations, sampling configurations, and mounting parameters are summarized in Table 4-3. Current stations that included directional wave measurements (entrance gauges – Stations 1, 2, and 3) were set to record 10-min averages, once every 20 min, and averages were centered on the top of the hour. ADPs configured with a digi-quartz pressure sensor (back-bay gauges – Stations 4, 5, and 13) were set to record 3-min averages, once every 6 min, and averages were centered at the top of the hour.

Currents were also incorporated in the directional wave measurements discussed in the wave measurement section. Three ADVOcean gauges were deployed at Stations 1, 2, and 3, and the sampling volume was located approximately 1 m above the seafloor.

Data reduction and quality control procedures for moored current meters

The current meter data were processed with a combination of manufacturer and in-house programs. Both ADPs and ADVOceans recorded in binary format. Following instrument retrieval, data files were downloaded and converted from binary to text (ASCII) file format. Subsequent data processing for the two instrument types involved different procedures as described in the following paragraphs.

Initial quality assurance/quality control (QA/QC) procedures for the ADP current meters were to generate amplitude data files for all beams and to inspect those files to verify that all current meters worked correctly. Pitch, roll, and heading data were then plotted to verify that the mooring remained upright and in place throughout the deployment. Each data file generated by SonTek software was then combined into one file to display the data as profiles. The ADPs were set to profile beyond the anticipated water depth to ensure that tidal fluctuations were included in the data. Profiles were edited to remove the erroneous data beyond the water depth by reference to both the pressure readings from the ADP or ADVOcean (if no pressure sensor was available on the ADP) and the recorded ADP beam amplitude data. The beginning and end of each file were removed for any data collected while the moorings were not deployed. Direction values were converted from degrees magnetic to degrees true north, decimal Julian dates calculated, and profiles vertically averaged. The vertically averaged profiles were then sent to the CHL FTP site.

The ADVOcean current meters were set to sample in burst mode for 4,096 data points at 2 Hz, once an hour. Bursts were averaged to obtain one reading per hour, and directions were converted from degrees magnetic to degrees true north. The beginning and end of each file were removed if data were collected while the instruments were not in the water. Averaged data were merged with the header files to add a time stamp to the record, and decimal Julian days were calculated prior to sending the data to the CHL FTP site.

For a final check of the data, vector time series plots were generated for each station location. Each plot contained vectors for the averaged ADP profile, the bottom ADP bin, and the averaged ADVOcean data where applicable (Stations 1, 2, and 3).

In addition to current measurements, the ADP at Station 1 had a MicroCAT installed, which measured conductivity and temperature. Temperature and conductivity values were recorded at the beginning of each sample interval and stored in the header file. Salinity values were calculated, the measurements were time stamped, and the data files transferred to the CHL FTP site.

General properties of the moored current meter data

Table 4-5 shows 28-day statistics for Deployments 1, 2, and 3 for both the ADP and ADVOcean current meters. One should note, for Deployment 1, the statistics calculated were for 15 days because of the short deployment length, and caution is warranted when comparing these values with the 28-day averages for Deployments 2 and 3. Data quality was good with two exceptions:
(a) Mooring 5 during Deployment 1 tipped over after day 3, and the current data are unusable following this event, and (b) the ADVOceans at Station 3 Deployment 1 and Station 1 Deployment 2 flooded, rendering the compass unusable. Data were corrected by using the compass values from the ADP mounted on the tripod above the ADVOcean, and incorporating the ADP compass values to calculate velocities referenced to earth coordinates.

Measurements of the current at the five stations within Willapa Bay show both site-specific and seasonal trends. Beginning with the seasonal trends, there was generally a small increase in the velocity with each successive deployment at all stations over the months of August through November.

As will be further discussed in the section, "Meteorology," measured wind speed increased over the months of August through November (Figures 4-22 through 4-25). Periods of stronger wind are evident in the current records. Three events of note occurred during 16-19 September, 1-4 October, and 12-17 November. All winds were from the south, producing northerly flowing currents. The exception to this was Station 5, which showed little or no visible influence of wind, but this station is well protected in the south end of the bay.

Direction of the current was site specific. The current at Station 1 had a north-south flow, exhibiting an alongshore flow pattern. Station 1 was also influenced by the prevailing wind. As the wind direction changed during the deployment months from northerly to southerly, the current flowed predominantly to the north in October and November. Directions of the current for Stations 2 and 3 were roughly mirror images of each other. Station 2 had a northwest to southeast flow pattern, and Station 3 had a southwest to northeast flow pattern. The measurements for both stations were strongly influenced by their location within the Middle Channel and their proximity to the surrounding shoals. Station 4, located near the entrance to Willapa River, had an east-west flowing current, which would be expected because of the orientation of the river. The current at Station 5 in Nahcotta Channel was directed southwest and northeast, which was consistent with the orientation of the channel.

Table 4-5
Statistics for Current Records

Deployment No.	Site No.	Start Date	End Date	Mean Temp deg C	Residual E-W Current cm/sec	Residual N-S Current cm/sec	Mean Speed cm/sec	Maximum Speed cm/sec	Residual Speed cm/sec	Net Direction deg True North	Total Variance cm^2/sec^2
1 (15 day)	ADP1	08/26/98	09/10/98	8.8	3.7	0.4	12.4	46.5	3.7	84	182
	ADV1	08/26/98	09/10/98	8.7	2.4	-0.6	8.3	33.0	2.5	103	94
	ADP2	08/26/98	09/09/98	11.3	-14.1	6.1	47.1	153.5	15.4	293	3,091
	ADV2	08/26/98	09/09/98	11.1	-12.4	4.1	42.9	152.1	13.1	288	2,565
	ADP3	08/26/98	09/09/98	11.6	6.7	-1.4	47.7	126.0	6.9	101	2,991
	ADV3	08/26/98	09/09/98	11.7	1.4	-2.2	41.1	109.0	2.6	147	2,181
	ADP4	08/26/98	09/09/98	15.1	3.7	1.6	37.8	93.4	4.0	67	1,897
	ADP5	08/26/98	09/09/98	Mooring turned over near beginning of deployment - no available data							
2 (28 day)	ADP1	09/11/98	10/08/98	10.9	-1.2	3.7	14.4	80.1	3.8	342	304
	ADV1	09/11/98	10/08/98	10.7	-1.0	0.3	9.6	69.4	1.0	285	180
	ADP2	09/11/98	10/07/98	12.3	-8.0	5.3	44.9	167.8	9.6	303	2,716
	ADV2	09/11/98	10/07/98	12.2	-6.0	4.7	29.2	137.1	7.7	308	1,317
	ADP3	09/11/98	10/07/98	12.6	8.7	-1.1	46.8	115.4	8.8	96	2,789
	ADV3	09/11/98	10/07/98	12.4	4.5	-2.3	36.3	88.1	5.0	117	1,673
	ADP4	09/10/98	10/05/98	14.4	-3.0	1.2	37.1	103.1	3.3	292	1,842
	ADP5	09/11/98	10/08/98	16.4	-0.2	-0.3	36.9	86.4	0.4	216	1,644
3 (28 day)	ADP1	10/10/98	11/06/98	12.8	0.3	4.8	16.2	65.4	4.8	4	331
	ADV1	10/10/98	11/06/98	12.7	-0.8	4.2	11.2	53.8	4.3	349	183
	ADP2	10/15/98	11/11/98	12.8	-8.5	9.8	51.5	196.1	13.0	319	3,478
	ADV2	10/15/98	11/11/98	12.6	-3.9	11.2	39.4	154.3	11.8	341	2,181
	ADP3	10/14/98	11/10/98	12.8	-2.9	-5.5	54.0	165.9	6.2	208	3,699
	ADV3	10/14/98	11/10/98	12.6	-4.3	-4.5	43.1	133.5	6.3	224	2,346
	ADP4	10/14/98	11/10/98	13.0	-5.3	0.9	38.4	122.4	5.4	280	2,008
	ADP5	10/15/98	11/11/98	12.6	-0.6	-1.4	34.9	87.2	1.5	202	1,483

ADCP transect cruises

Over-the-side Doppler current measurements were collected in mid-November 1998 during a spring-tide condition. Measurements of the current were made in conjunction with the profiling CTD cruise. The purpose of the over-the-side current measurements was to obtain discharges across the various channels at the mouth and head of the bay during both ebbing and flooding tides. The discharges indicate the net movement of water into and out of the bay. In addition, the transect measurements provide information on the regional variation of the current structure across the channels.

An RDI Broadband 600-kHz ADCP was mounted on the port side of a vessel during transects across the bay. Navigation information was obtained with a Trimble GPS station with differential capabilities. Configuration of the ADCP for the cruise is detailed in Table 4-6. Setup of the instrumentation occurred on 16 November. Communication with all systems was verified, and configuration files were created.

Table 4-6
ADCP Configuration Parameters

Parameter	Value
Cell (bin) size	0.5 m
Blank after transmit cell size	0.5 m
Pings per ensemble	4
Time between pings	0 sec
Average (ensemble) interval	4 sec
Magnetic offset	19 deg
Maximum bottom tracking depth	30 m

Transect cruises were conducted on 17-18 November 1998. Current data were collected in three geographic regions displayed in Figure 4-8. Transects in the south bay across Nahcotta and Stanley Channels were obtained on 17 November. Fifteen transects were measured across the south bay (Figure 4-9). Visibility was low during the first 2 hr of data collection because of fog. Difficulties were also encountered during the first day when navigation data were not collected by the data acquisition software. Bottom tracking was functional, so ship speeds were still subtracted from the recorded current measurements. As a backup for the lost navigation data, time and ship position displayed on the GPS were handwritten in a field log at approximately 5-min intervals. Positions were recorded by hand along all transects during the remainder of the cruise. Total discharge as reported on the display monitor was noted in the field log for each transect.

Transects 1 through 12 were traversed during the flood tide, and Transects 13 through 16 were traversed on the ebb tide (note: there is no Transect 2). Transits across the south bay on the ebb tide were frequently split into two transect files because of the presence of a shoal between the two channels. The depths over the shoal were generally too shallow for the ADCP to collect data. Transects on the flood tide were located further north than the ebb-tide transects because the water at the original transit area was too shallow for the boat to safely operate without grounding. Because of the northern location of the flood tide transects, measurements were possible across the complete channel without having to split the transect into two separate files.

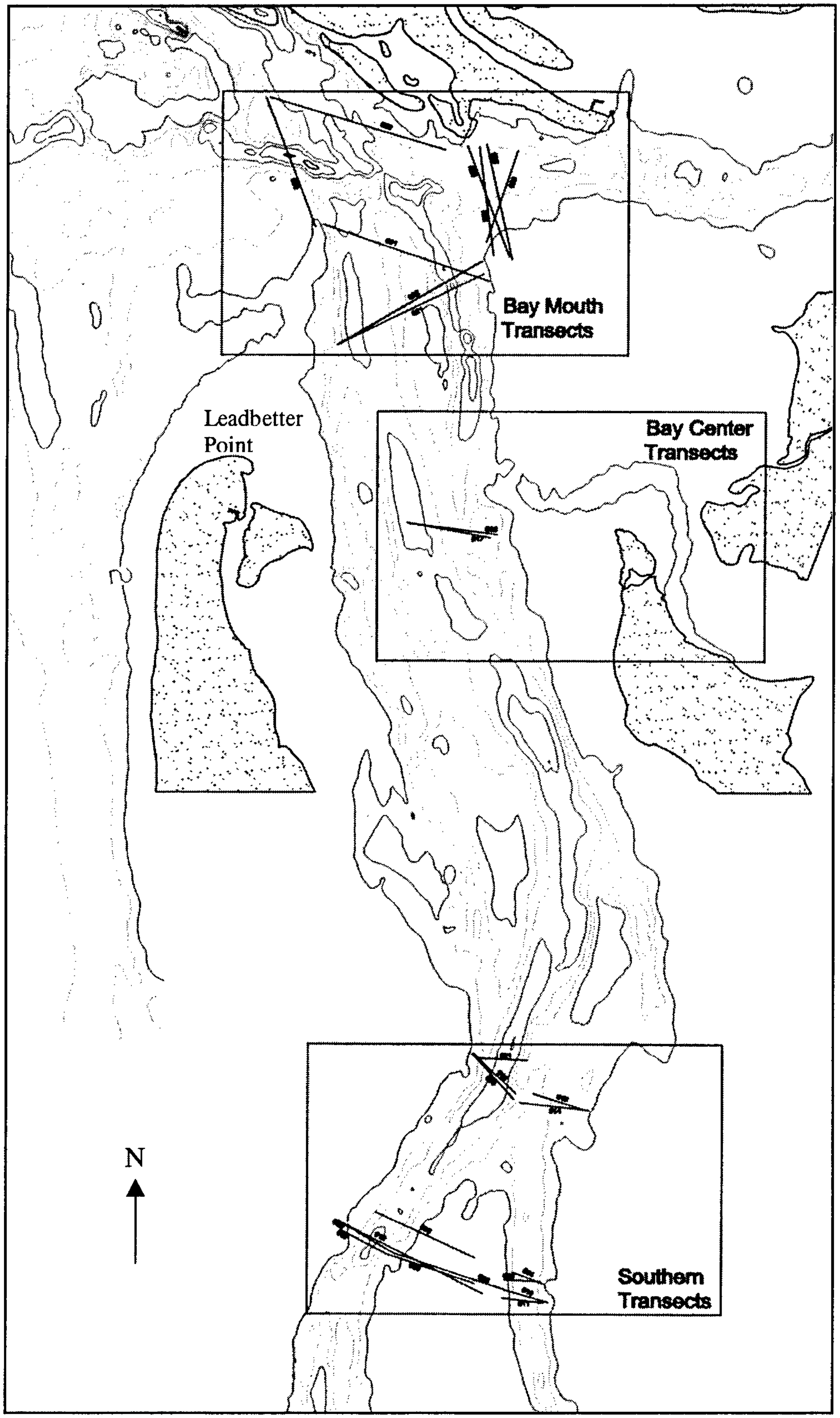

Figure 4-8. ADCP transects in Willapa Bay, 17-18 November 1998

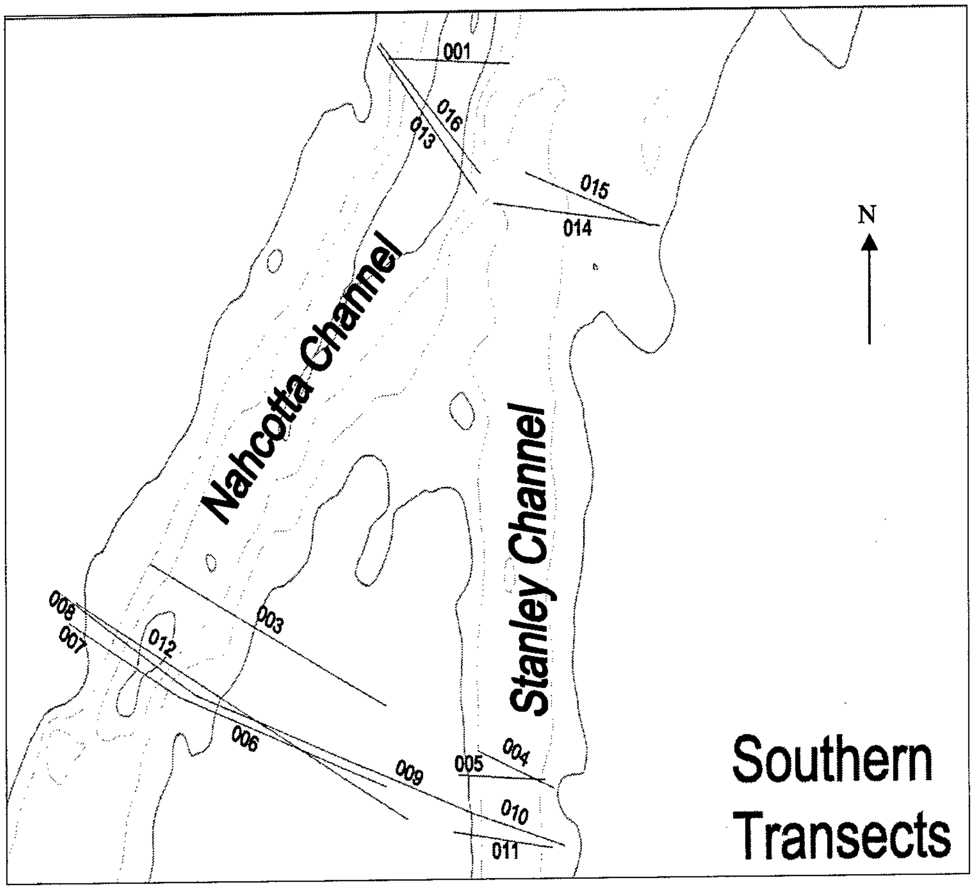

Figure 4-9. ADCP transects in southern Willapa Bay, 17 November 1998

The final two transects on 17 November (17 and 18) were made outside the entrance to Bay Center (Figure 4-10).

Transects near the mouth of the bay were obtained on 18 November. Nine transects were made on this date, five on the morning flood tide and four on the afternoon ebb (Figure 4-11). Because of the wave action and exposed shoals during the early part of the flood tide, transects were completed across the mouth of the Willapa River (Transects 23, 25, and 26) and across the entrance to the southern channels (Transects 21 and 22). On the ebb tide, with higher water levels and calmer seas, a box pattern of transect measurements was obtained at the mouth of the bay (Transects 29 through 32). Problems were encountered occasionally with bottom tracking during times of stronger current. When bottom tracking was lost, navigation was recorded; thus, ship motion was continually subtracted from the measured currents.

Data reduction for ADCP transect cruises

Data were collected and partially processed with RDI Transect software that accompanies the ADCP. ASCII profiles created in Transect were further manipulated with EHI software to produce depth-averaged vectors with location fixes for each profile.

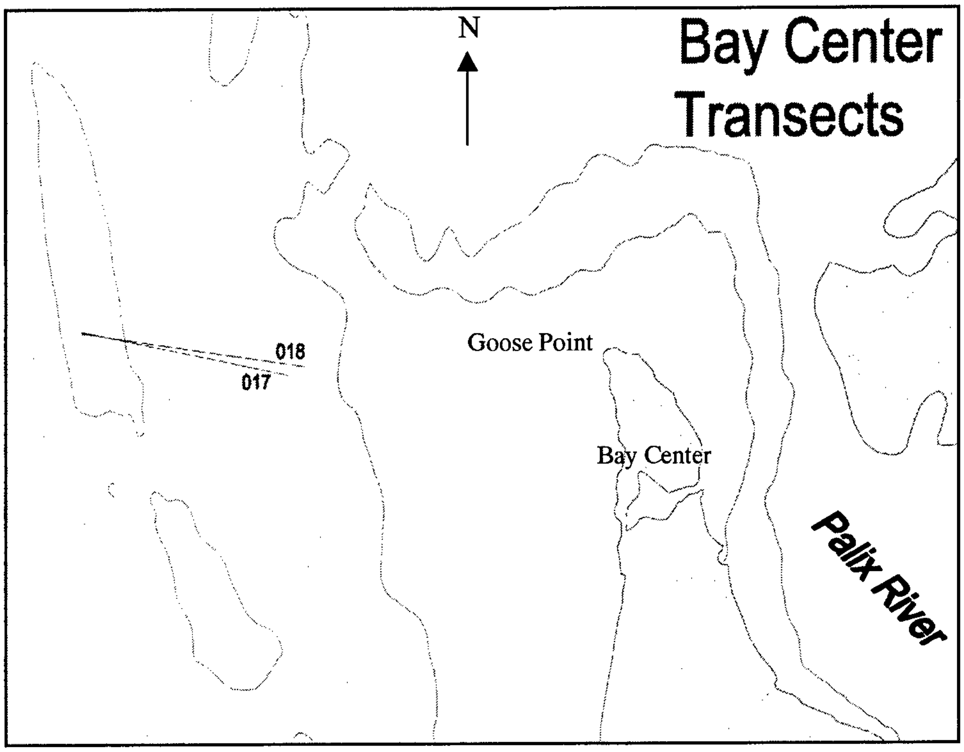

Figure 4-10. ADCP transects near Center Channel, Willapa Bay, 7 November 1998

RDI Transect calculates values for discharge along a channel section while the ship is underway. These values were recorded in the log at the conclusion of each channel-crossing transect. Two methods were used to fix the profile locations. Transects 1 through 23 use bottom track information to find location. Transects 25 through 32 use GPS coordinates to record location.

For Transects 1 through 23, current speed was obtained by removing the bottom-track velocity, but no coordinate stamps were recorded in the profiles. Transect software records bottom-track displacement north (ΔN) and displacement east (ΔE) with each profile. Consulting logged coordinates for the beginning of each transect made correcting possible by the ΔN and the ΔE values recorded for a profile to arrive at a location for that profile.

Transects 25 through 32 had GPS coordinate stamps in the ADCP profiles. These coordinates were read for each profile and reissued with the depth-averaged values in the final output. Speed and direction were plotted for each transect to verify that software output conformed to expected values. Attention was focused on transects with reciprocal tracks to verify that ADCP head-alignment corrections were accurate, giving similar current profiles on all courses.

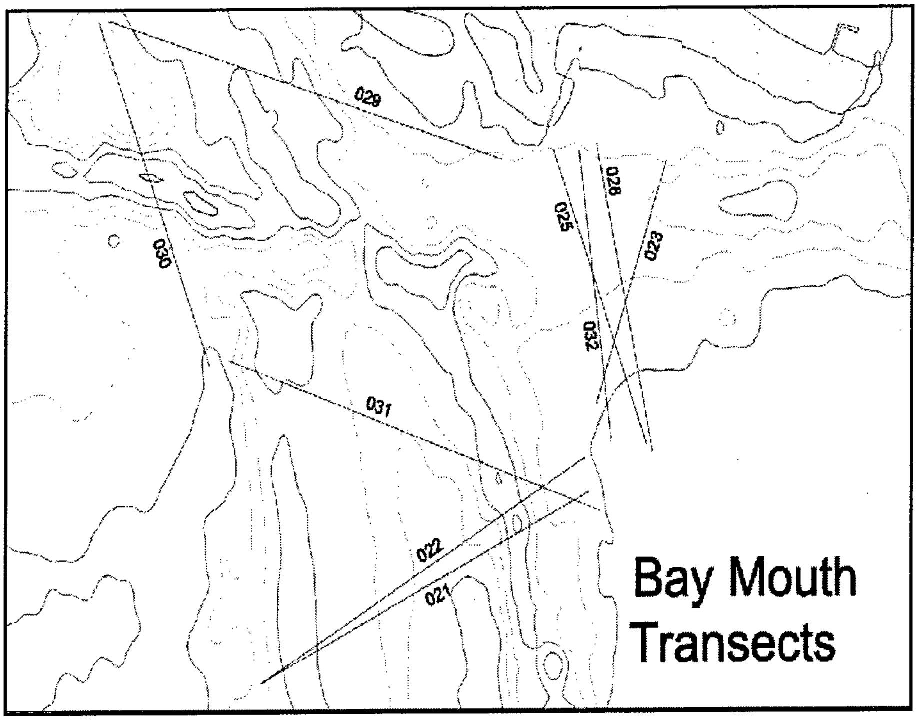

Figure 4-11. ADCP transects at entrance of Willapa Bay, 18 November 1998

General properties of the ADCP transect data

As noted in the “ADCP transect cruises” section, the ADCP transects were collected during a spring tide the week of 16 November (Figure 4-12). Figure 4-13 provides an expanded view of the water-level variation for the data collection days, with markers indicating at what tidal stage each transect was measured. For both days, approximately the same number of transects were traversed on the flood tide as for the ebb tide. Transport and regional current structures for each measurement day are reviewed in the following paragraphs.

Discharge is defined as the total volume of water flowing through a cross section. The ADCP measures discharge in cubic meters per second for each ensemble and yields a sum of the discharge for each transect. Table 4-7 lists the summed discharge values for each transect measured at the southern end of the bay. Negative discharge indicates southward or flooding flows, and positive discharge indicates northward or ebbing flows. For the location of each transect, refer to Figure 4-9. The tidal height referenced to meters mllw at the beginning of each transect is also listed in Table 4-7.

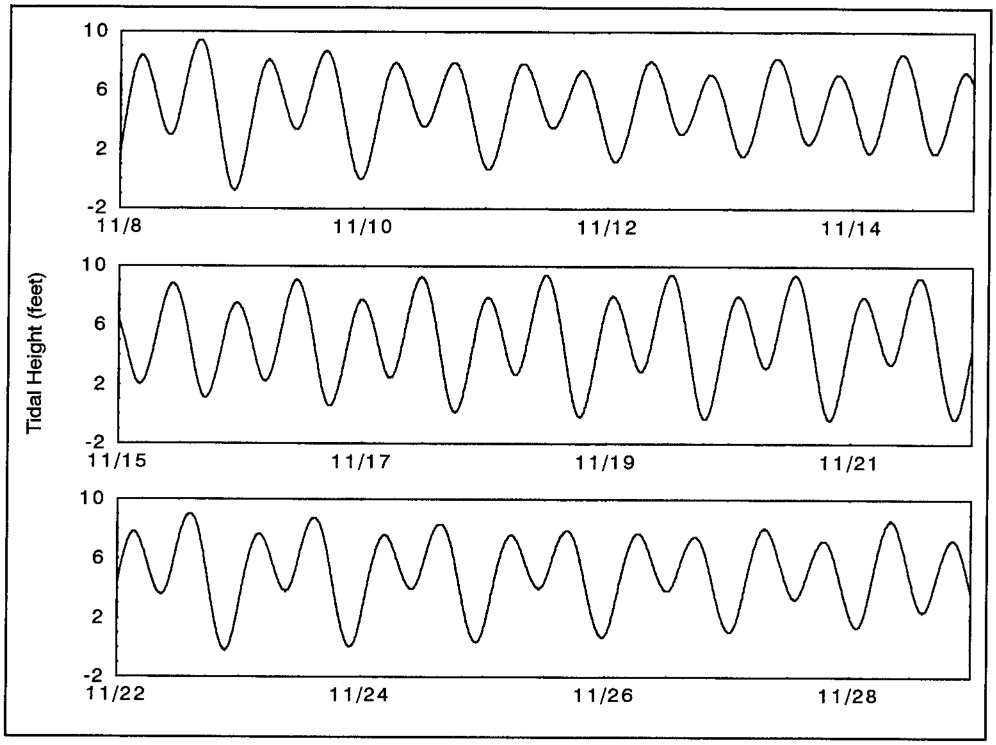

Figure 4-12. Predicted tide for 3 weeks in November 1998

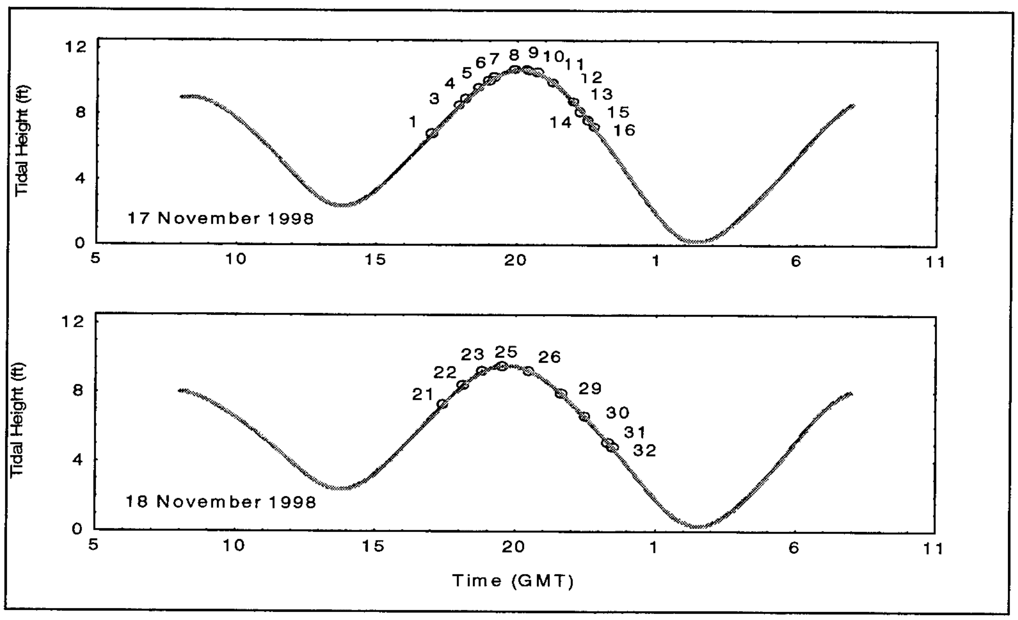

Figure 4-13. Water level for ADCP transect collection days (circles denote tidal stage each transect was collected, and numbers refer to transect number)

Table 4-7 Discharge Values for South Willapa Bay			
Transect Number	**Location Description**	**Discharge, cu m**	**Tidal Height, m (mllw)**
1	South Channel	-9,407	2.07
3	Nahcotta Channel	-7,585	2.59
4	Stanley Channel	-3,781	2.71
5	Stanley Channel	-3,340	2.93
6	Shoal Area	-990	3.05
7	Nahcotta Channel	-3,472	3.11
8	Nahcotta Channel	-2,320	3.26
9	Shoal Area	870	3.26
10	Stanley Channel	-61	3.23
11	Stanley Channel	664	3.20
12	Nahcotta Channel	6,404	3.02
13	South Channel	8,718	2.65
14	South Channel	5,802	2.47
15	South Channel	5,873	2.32
16	South Channel	9,801	2.19

Discharges summed across all channels on the mid-ebb and flood tides were approximately equal. For example, Transects 1 and 16 were collected near the same location at roughly the same stage on the flooding and ebbing tides, respectively. The sums of other transects indicate this same result. The total for Transects 5, 6, and 7 on the flooding tide equals –7,802 cu m/sec, and the total for Transects 11 and 12 on the ebbing tide was +7,068 cu m/sec.

The shoal area between Nahcotta and Stanley Channels showed smaller discharges than expected. It also exhibited the first ebbing discharge values (Transect 9) while the two channels were still flooding (Transects 8 and 10).

Depth-averaged current speed across Stanley Channel ranged between 10 and 60 cm/sec, with peak speed reaching about 70 cm/sec. The current range had less variability for the Nahcotta Channel at 30-50 cm/sec with peaks reaching about 75 cm/sec. Current speed to the north of both channels ranged from 40 to 80 cm/sec with peaks reaching 100 cm/sec.

Table 4-8 lists the summed discharge values for the transects measured at the north end of the bay. Transect locations are displayed in Figure 4-11.

Table 4-8 Discharge Values for North Willapa Bay			
Transect Number	**Location Description**	**Discharge, cu m**	**Tidal Height, m (mllw)**
21	Southern Mouth	-27,042	2.23
22	Southern Mouth	-28,079	2.56
23	Willapa River	-8,872	2.80
25	Willapa River	-6,308	2.90
26	Willapa River	1,033	2.80
29	Outer Mouth	14,472	2.41
30	Outer Mouth	11,520	2.01
31	Southern Mouth	23,008	1.55
32	Willapa River	10,530	1.46

Discharge at the mouth of the bay shows the same directions as that for the southern bay, outbound on the ebb tide and inbound on the flood. One major difference was the magnitude, which showed a factor of 10 increase. Comparisons between the ebb- and flood-tide discharges were more difficult to

make for this area because of the inability to occupy all transect areas during the same phase of the tide. However, some general comparisons can be made. For example, discharges for Transects 21 and 22 compared reasonably well with discharge for Transect 31. Similar comparisons can be made between Transects 23 and 26 at the mouth of the Willapa River.

Increases in the current at the bay entrance as compared with the southern transects of the previous day were also evident. Current speed on transects across the southern mouth ranged between 50 and 120 cm/sec. Current speeds across the mouth of the Willapa River were in the range of 20 to 100 cm/sec, averaging around 50 cm/sec. The least variable current speeds across transects were seen at the outer mouth area. Current speed for Transects 29 and 30 ranged between approximately 60 and 80 cm/sec.

Water Level and In Situ Conductivity and Temperature Measurements

Much of the water exchange between Willapa Bay and the Pacific Ocean is tidally driven, and any quantification of the flow in the bay will depend upon the predicted tide level throughout the bay.

Instruments and field procedures

Conductivity, temperature, and pressure data were collected using Sea-Bird Electronics, Inc., SBE 26-03 tide gauges. Tide gauges were installed at Stations 6, 7, 8, and 9 (Figure 4-1 and Table 4-9).

Gauges were set to record 3-min averages every 3 min. Three-minute averages were synchronized to logging times of the four gauges to begin at 01:30 min before the hour so that the first average each hour centered at the top of the hour.

Table 4-9
Water Level Station Locations and Measured Parameters

Station Number	Location Description	Latitude deg N	Longitude deg W	Elevations Relative to Toke Point, m (mllw)	
				Sensors	Bay Floor
6	South Bend	46 40.110	123 48.771	-1.276	-1.65
7	Nahcotta	46 30.098	124 01.430	-0.632	-1.15
8	Naselle River	46 25.794	123 54.374	-1.674	-2.13
9	Bay Center	46 37.404	123 56.844	-1.762	-3.13

Gauges were mounted on pilings in swinging pipe brackets that allowed servicing without diving. Work on the gauges was done in small boats by swinging the pipe up and away from the base of the piling. After data download and battery change-out, the pipe was returned to vertical with the gauge reoccupying the elevation prior to servicing. All tide gauge mounts performed well over the duration of the project with no slippage relative to the pilings.

Data reduction

Water levels at Stations 6, 7, 8, and 9 were derived from subsurface pressure records. This derivation was made using the United Nations Educational, Scientific and Cultural Organization (UNESCO) 1983[1] equation of the state for seawater density. Water level was then referenced to Toke Point mllw datum using offsets measured during a leveling survey of the gauge locations relative to National Geodetic Survey benchmarks.

Tide gauge water level

Total pressure (barometric pressure plus water pressure) was recorded at each water-level station, and barometric pressure was logged hourly at the EHI meteorological station in the Bay Center Channel. During data processing, atmospheric pressure values were subtracted from the recorded water-level data.

The equation of state of seawater was used to calculate local seawater density in the area of the gauge based upon measured gauge pressure, salinity, and temperature. Depth of the gauge was calculated from density, pressure, and gravitational acceleration for the 40°N latitude.

Survey to Toke Point datum

The leveling survey accessed in this study was performed by the Washington Department of Ecology (WDOE).[2] In summary, water level at marked times was surveyed to the North American Vertical Datum of 1988 (NAVD 88) vertical datum by means of Real-Time Static GPS. Observations were taken to National Geodetic Survey (NGS) 2-cm standards using two control stations for each site. Each site was occupied twice over consecutive days.

An offset from NAVD 88 to Toke Point mllw was established by consulting published National Ocean Service (NOS) and NGS data sheets. The end product of this survey was water-level height to Toke Point mllw established at each site for a known time.

Gauge water level to Toke Point datum

By taking the water-level record from a tide gauge for the time of the elevation water-level survey to Toke Point mllw, it was possible to create an individual offset for each tide gauge, correcting the gauge-derived water level to a Toke Point mllw reference. This correction based on observed conditions at a known time was unique to each gauge installation and provided consistent reference regardless of hydraulic conditions within the individual pressure cell.

[1] Fofonoff, N. P., and Millard, Jr., R. C. (1983). "Algorithms for computation of fundamental properties of seawater," UNESCO technical papers in *Marine Science* 44.
[2] Daniels, R. (1998). Letter report to Evans-Hamilton, Inc. on the Washington Coastal Gauge Control Project as part of a U.S. Army Corps of Engineers study on Willapa Bay, p. 14.

With this process, it was possible to reference all four tide measuring gauges to an existing mllw datum at Toke Point with the assumptions that water density above the gauge was homogenous and that barometric pressure was stable over a 60-min period.

Data trends

Data from all water-level gauges were plotted after processing to observe trends and check quality. Figure 4-14 shows a sample plot of Deployment 4 at Station 9.

Processed data showed strong tidal signals in salinity and temperature at river Stations 6, 8, and 9. Station 7 in Nahcotta Harbor showed less variance in salinity and temperature.

Unknown factors periodically dampened tidal fluctuations in salinity at Station 6 in South Bend. Salinity became stable over several days and then returned to a tidally varying signal. Sensor malfunction is not the most likely source of unusual stability followed by a strong variance in amplitude. Other sources of this variance may be saline discharge from an adjacent oyster-processing industry or biological activity interfering with the flow of water through the conductivity cell. Little bio-fouling was noted on the water-level gauges at all stations.

Profiling CTD Survey

Purpose of data collection

Profiling Conductivity Temperature Depth (PCTD) data were collected to determine the vertical structure of the water column within Willapa Bay. PCTD casts were taken on high- and low-slack tides during a spring tidal sequence.

Instruments and field procedures

Salinity data were collected with a Sea-Bird Electronics, Inc., SBE-19 Profiler. The conductivity cell, the temperature sensor, and pressure gauge had been calibrated within the preceding year. This instrument derived salinity from conductivity measurements; likewise, density was calculated from temperature and salinity.

All stations were sampled on both a high-slack and low-slack tide with the exception of Station 11, which was not sampled on a low-slack tide because of weather constraints. Pertinent station information is listed in Table 4-10. Station position was determined by means of differential GPS. In addition to stations within the bay channels, PCTD measurements were taken at the tide gauges. Once on station, the instrument was soaked at the water surface for roughly 3 min to allow the sensors to equilibrate. The instrument was then lowered through the water column at a rate of approximately 20 cm/sec, allowed to gently touch bottom, and then raised back to the surface. Data were downloaded at each station and checked in the field for data quality.

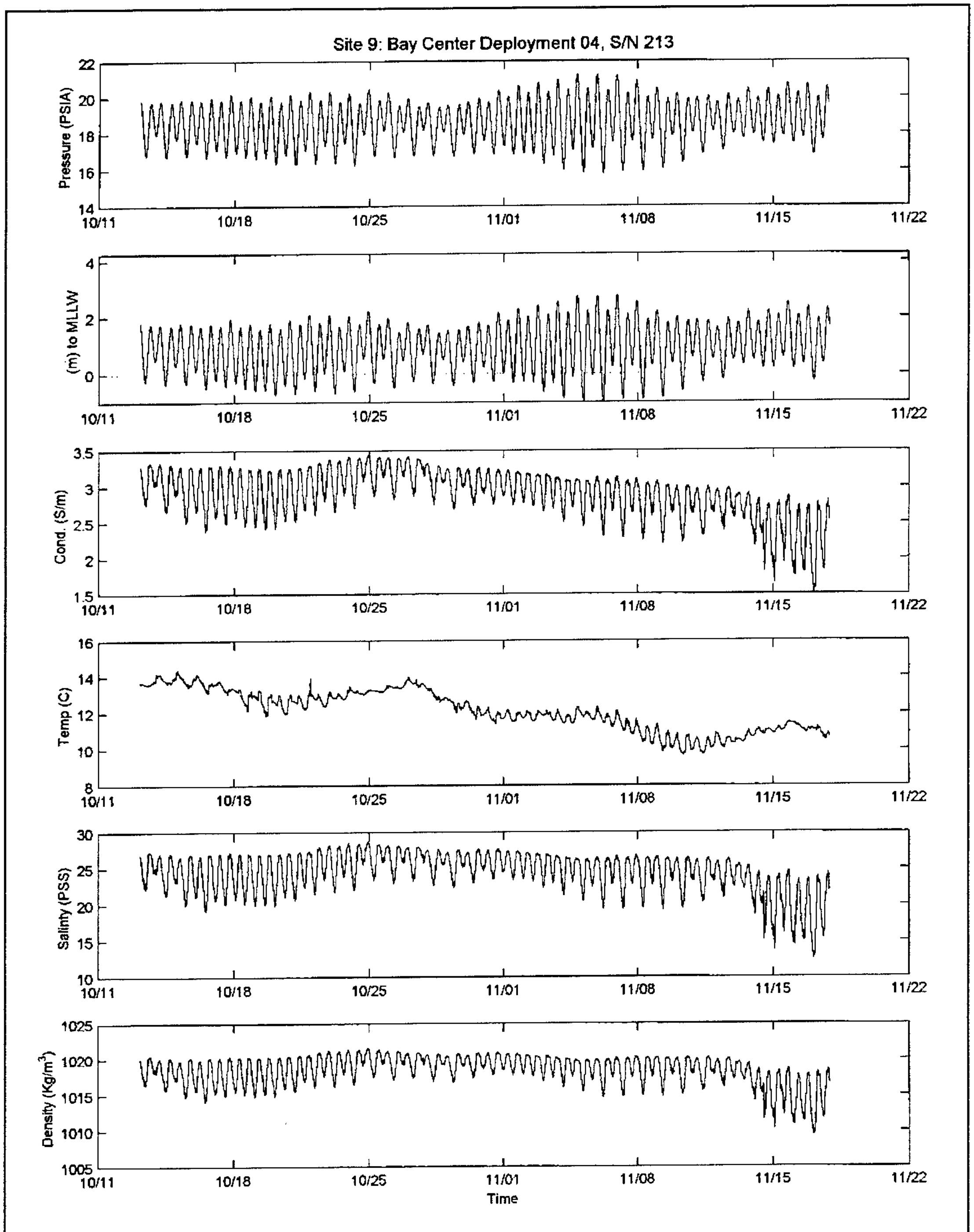

Figure 4-14. Example of tide gauge data, Station 9, Deployment 4

Table 4-10
Profiling Conductivity Temperature Depth (PCTD) Survey Information

Date	Time (downcast) GMT	Julian Day (downcast) GMT	Station No.	Tide	Latitude deg N	Longitude deg W	Maximum CTD Depth, m	Echosounder reading, m
11/17/98	15:28:37	321.6448669	1	Low	46 40.133	123 48.710	6.5	7.0
11/17/98	18:46:24	321.7822164	1	High	46 40.133	123 48.710	9.0	9.1
11/17/98	15:11:07	321.6327141	2	Low	46 41.178	123 49.066	10.0	10.1
11/17/98	18:57:10	321.7896991	2	High	46 41.169	123 49.059	10.5	11.6
11/17/98	14:59:01	321.6243229	3	Low	46 42.447	123.50.884	8.5	8.5
11/17/98	19:10:13	321.7987616	3	High	46 42.454	123 50.891	10.5	10.4
11/17/98	14:35:04	321.6076852	4	Low	46 42.160	123 53.356	7.5	7.9
11/17/98	19:23:26	321.8079398	4	High	46 42.167	123 53.333	9.5	9.8
11/17/98	14:03:30	321.5857639	5	Low	46 41.606	123 55.678	6.0	13.5
11/17/98	19:37:48	321.8179109	5	High	46 41.617	123 55.655	9.0	9.1
11/16/98	20:45:51	320.8651736	6	High	46 41.895	123 58.128	13.5	14.0
11/16/98	23:06:53	321.0464468	6	Low	46 41.916	123 58.097	11.0	11.4
11/16/98	19:30:25	320.8127836	7	High	46 42.285	124 01.636	14.0	13.4
11/17/98	00:49:20	321.0342593	7	Low	46 42.270	124 01.609	11.0	11.6
11/16/98	19:55:25	320.8301447	8	High	46 43.436	124 03.718	25.0	27.4
11/17/98	00:34:06	321.0236806	8	Low	46 43.368	124 03.621	20.5	21.9
11/16/98	19:10:36	320.4656887	9	High	46 41.879	124 03.012	12.5	14.0
11/17/98	00:14:51	321.0103067	9	Low	46 41.843	124 03.121	10.0	10.1
11/16/98	18:30:26	320.7711285	10	High	46 40.382	124 00.775	9.5	9.8
11/16/98	23:50:18	320.9932581	10	Low	46 40.400	124 00.814	8.0	7.9
11/16/98	17:45:26	320.7398843	11	High	46 37.680	123 59.783	11.5	12.5
11/18/98	15:31:01	322.6465451	12	Low	46 35.065	123 58.068	16.5	16.5
11/18/98	19:56:09	322.8306597	12	High	46 35.052	123 58.060	18.0	18.1
11/18/98	15:51:02	322.660434	13	Low	46 33.546	123 56.756	10.0	10.4
11/18/98	20:11:56	322.8416204	13	High	46 33.531	123 56.758	12.0	11.6
11/18/98	15:14:15	322.63489	14	Low	46 32.067	123 59.025	14.0	14.3
11/18/98	20:36:25	322.8593171	14	High	46 32.072	123 59.060	16.0	16.5
11/18/98	16:19:24	322.6801331	15	Low	46 30.739	123 58.319	9.5	9.8
11/18/98	19:28:07	322.8111921	15	High	46 30.745	123 58.325	11.0	11.6
11/18/98	13:53:55	322.5791088	16	Low	46 30.096	124 01.431	3.0	3.0
11/18/98	21:37:09	322.9007986	16	High	46 30.103	124 01.430	5.0	4.0
11/18/98	14:31:20	322.6050926	17	Low	46 29.150	124 00.830	14.0	14.3
11/18/98	20:56:41	322.8726968	17	High	46 29.165	124 00.852	15.5	16.2
11/18/98	16:31:42	322.6886748	18	Low	46 29.192	123 57.797	11.5	11.6
11/18/98	19:14:41	322.8018634	18	High	46 29.172	123 57.795	7.5	13.1
11/18/98	16:44:24	322.6975	19	Low	46 27.978	123 56.486	17.0	17.4
11/18/98	19:00:12	322.7918056	19	High	46 27.935	123 56.453	19.5	13.1
11/18/98	14:48:49	322.617228	20	Low	46 26.723	124 00.300	15.0	14.9
11/18/98	21:11:48	322.8831887	20	High	46 26.738	124 00.301	17.0	17.7

Data reduction

All data were processed with Sea-Bird software. Upcast data and data recorded while soaking were discarded, leaving the downcast data. Temperature, salinity, and density data were then averaged into 0.5-m bins to produce a profile corresponding to saltwater depth. These data were then averaged over the water column to produce a single datum. The profiles were graphed in Figure 4-15 through 4-21 to illustrate the water column structure and to verify data quality. In these figures, the plotted variable "PSS" represents practical salinity scale units, parts per thousand. The density is presented in σ_t values where σ_t = (water density -1×10^3 g/cm^3).

Data properties

The salinity and density structure for the Willapa River (Stations 1 through 5) greatly differed between high and low tide (Figures 4-15 and 4-16). The freshwater lens was much larger throughout the river during the low slack than on the high slack. This lens persisted somewhat at Station 1 (South Bend) on the high slack, but was less than 2 m thick for the remaining stations. Temperature throughout the water column did not change.

Unlike the Willapa River stations, the vertical structure for sites within the central portion of the bay (Stations 12 through 14) remained largely unchanged between high- and low-slack tides. Although the profile shape did not change, the salinity during low slack was generally lower than on high slack (Figure 4-17). Salinity levels at stations in the Naselle River channel (Stations 15, 18, 19) varied between the tidal stages; much like the Willapa River stations, salinity was lower throughout the water column on the low tides (Figure 4-18). High-slack measurements showed little stratification within the water column. Nahcotta channel stations (Stations 16, 17, 20) showed little variation between high- and low-slack tides (Figure 4-19). Water temperature remained constant at all stations.

Vertical profiles for stations located in the northern portion of the bay (Stations 6 through 11) changed relative to their proximity to the Willapa River (Figures 4-20 and 4-21). Salinity and density values at Stations 6, 7, and 8 showed a lens on the low slack that was greatly diminished (Station 6) or gone (Stations 7 and 8) on the high slack. Stations farther removed from the mouth of the Willapa River showed little or no change between the two tidal stages. As with all other stations, temperature did not vary.

Meteorology

Instruments and field procedures

A Campbell Scientific, Inc., MetData 1 weather station was installed 12 August 1998 on the Bay Center Channel Light, Number 2 platform near the center of the study area (Figure 4-1). The MetData 1 weather station measured wind speed, wind direction, air temperature, and barometric pressure approximately 10 m above the water surface. All data were stored in a CR10 data-logger and retrieved approximately once a month (Table 4-1) using a portable computer.

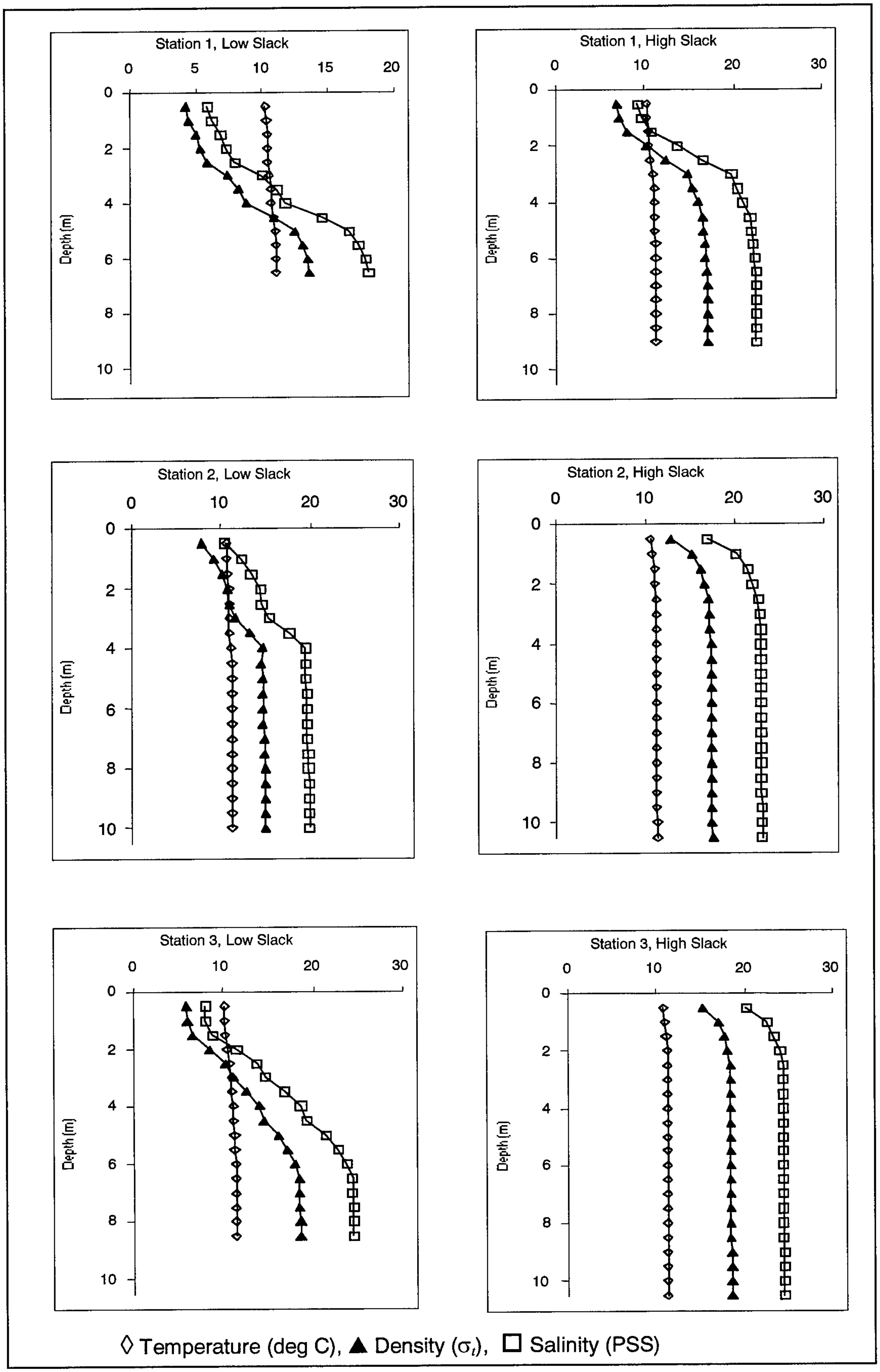

Figure 4-15. Temperature, density, and salinity profiles for stations in Willapa River during low- and high-slack tides (Stations 1-3)

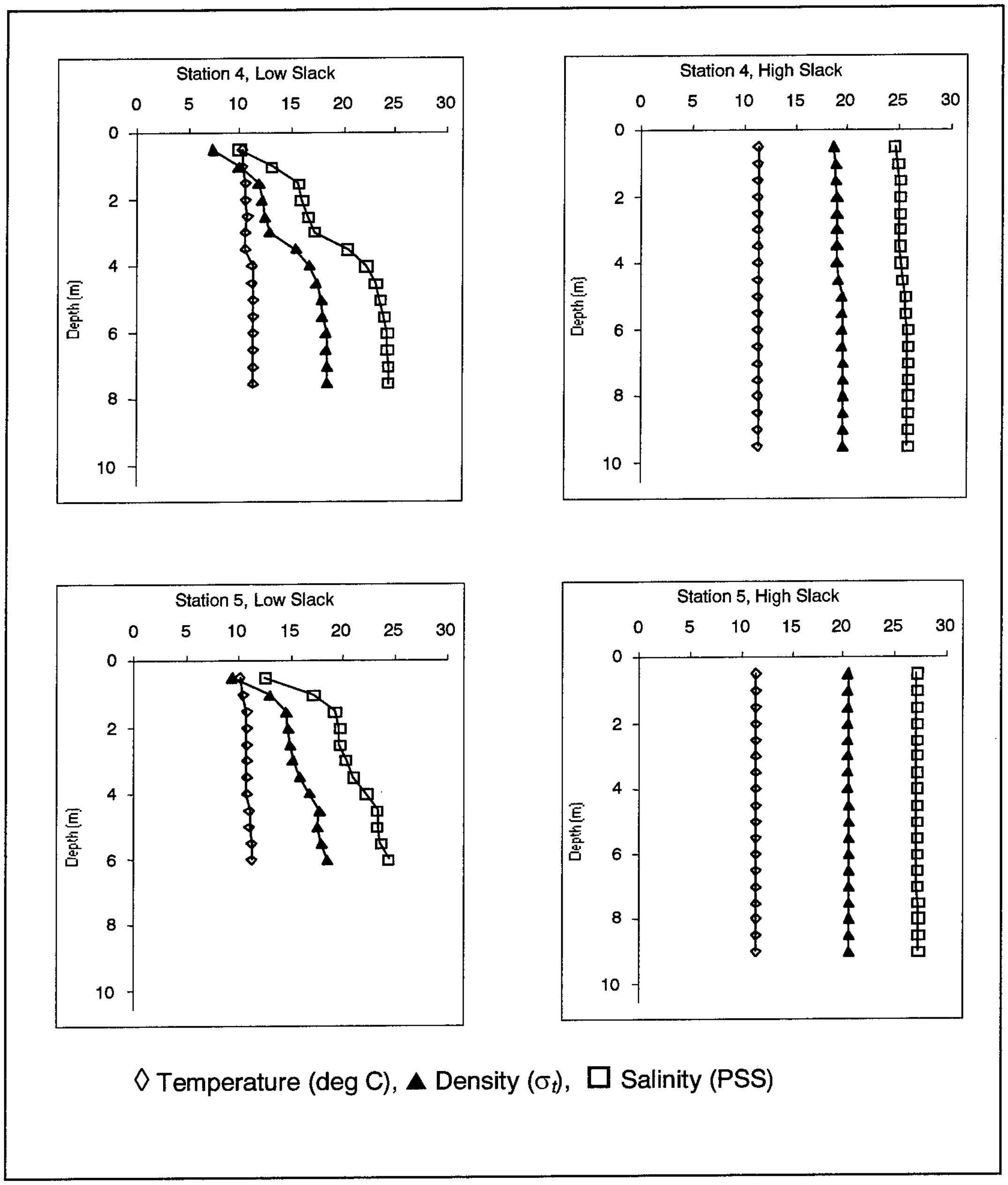

Figure 4-16. Temperature, density, and salinity profiles for stations in Willapa River during low- and high-slack tides (Stations 4 and 5)

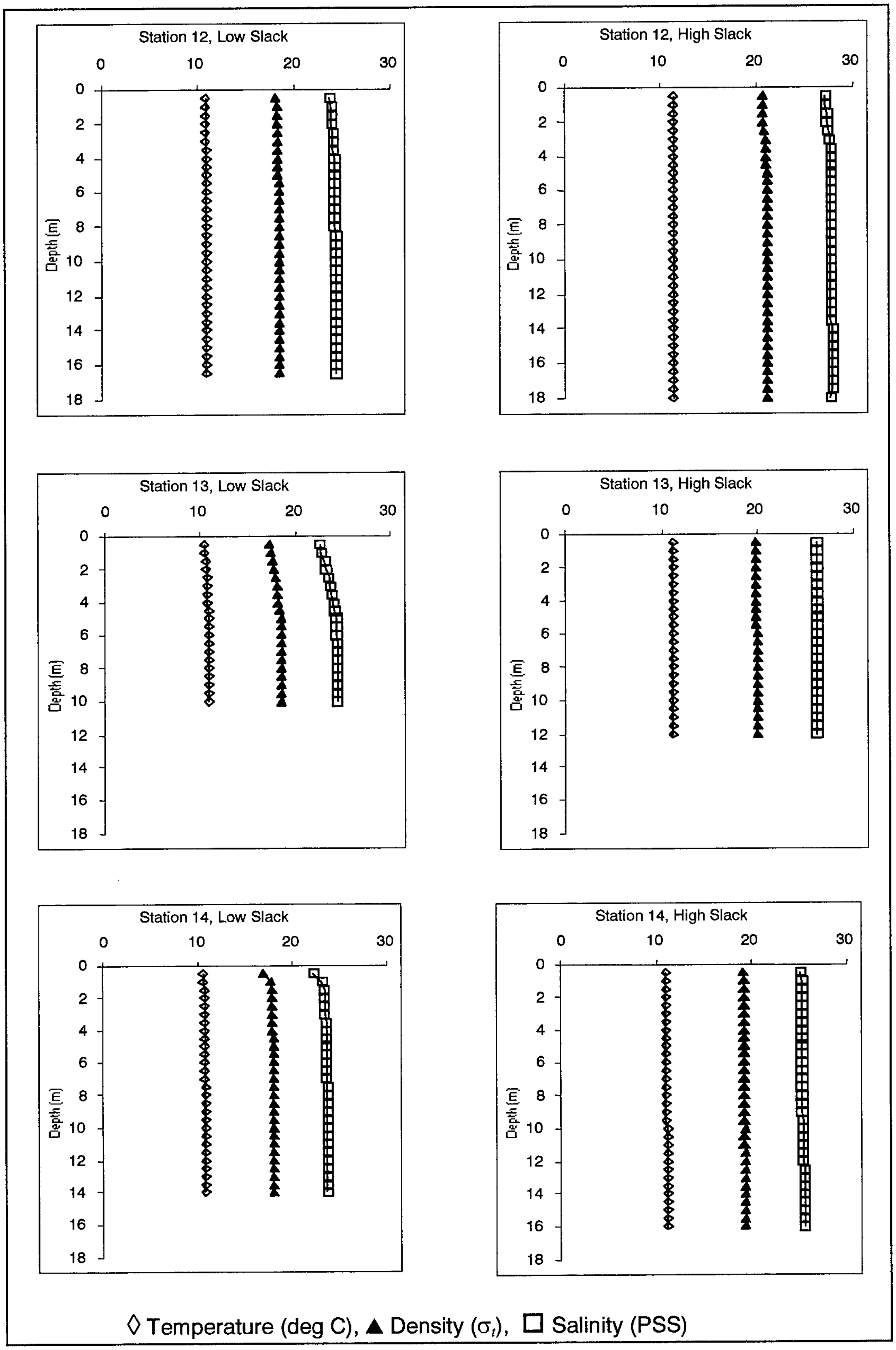

Figure 4-17. Temperature, density, and salinity profiles for stations in central Willapa Bay during low- and high-slack tides (Stations 12-14)

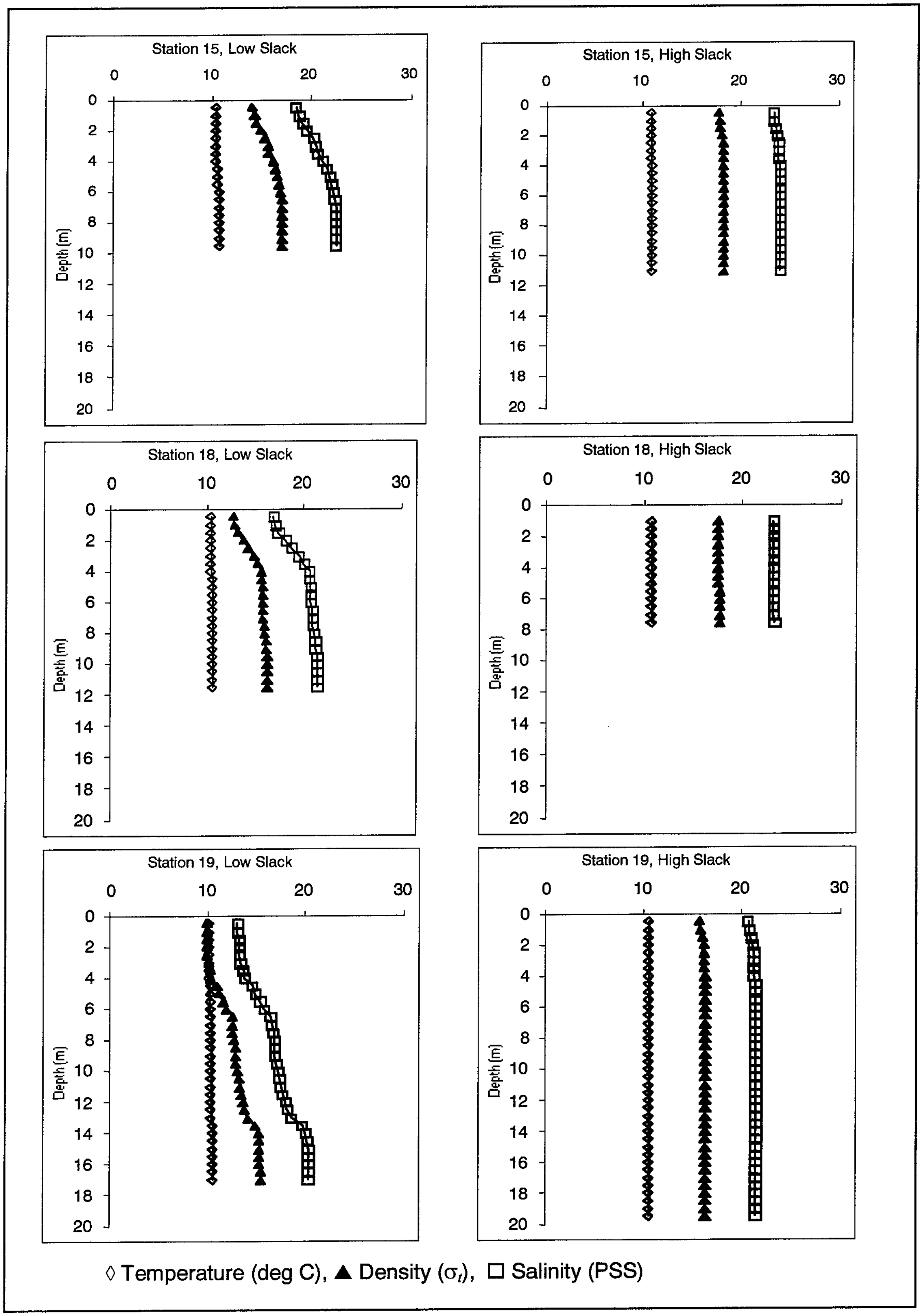

Figure 4-18. Temperature, density, and salinity profiles for stations in Naselle River Channel during low- and high-slack tides (Stations 15, 18, and 19)

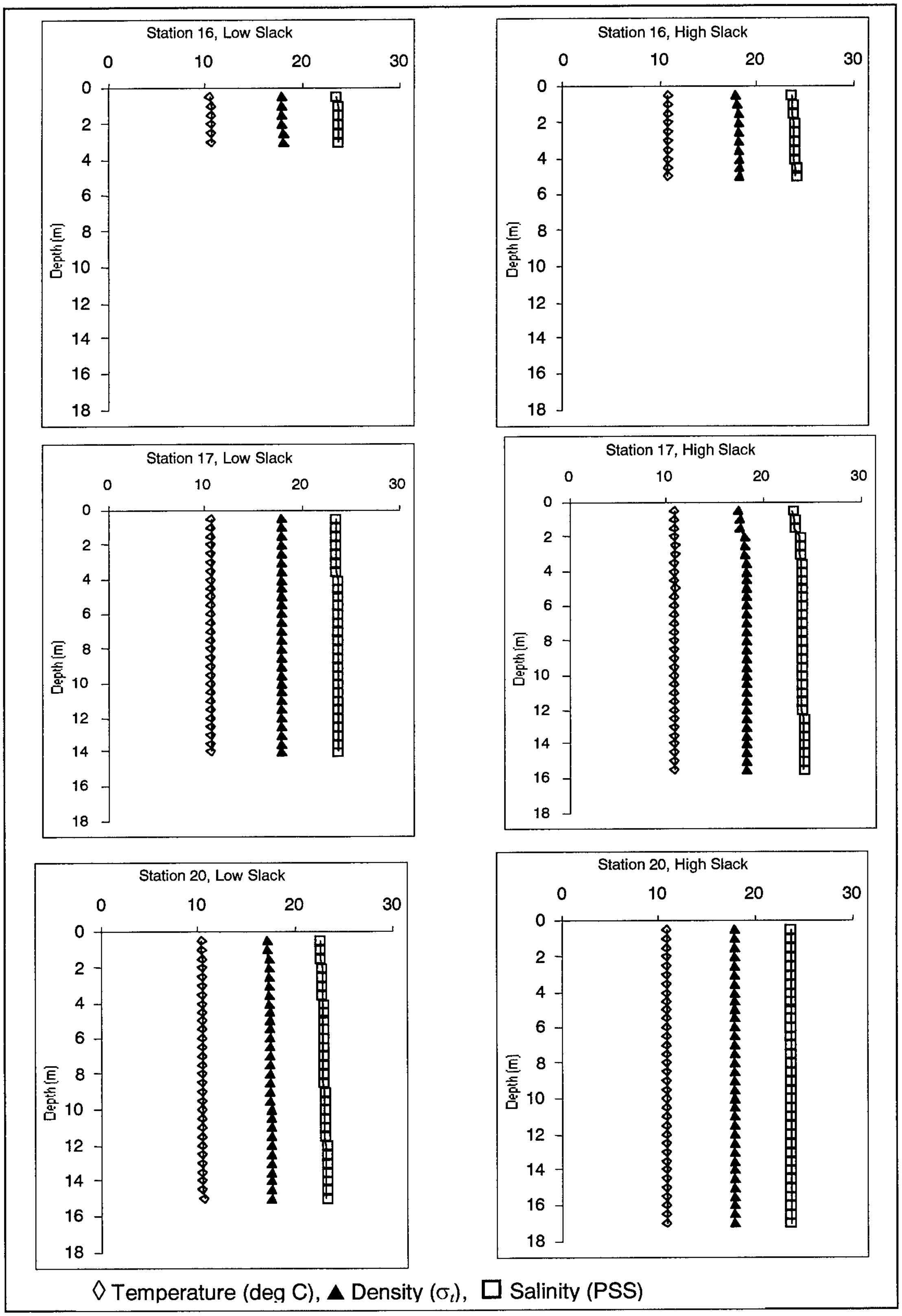

Figure 4-19. Temperature, density, and salinity profiles for stations in Nahcotta Channel during low- and high-slack tides (Stations 16, 17, and 20)

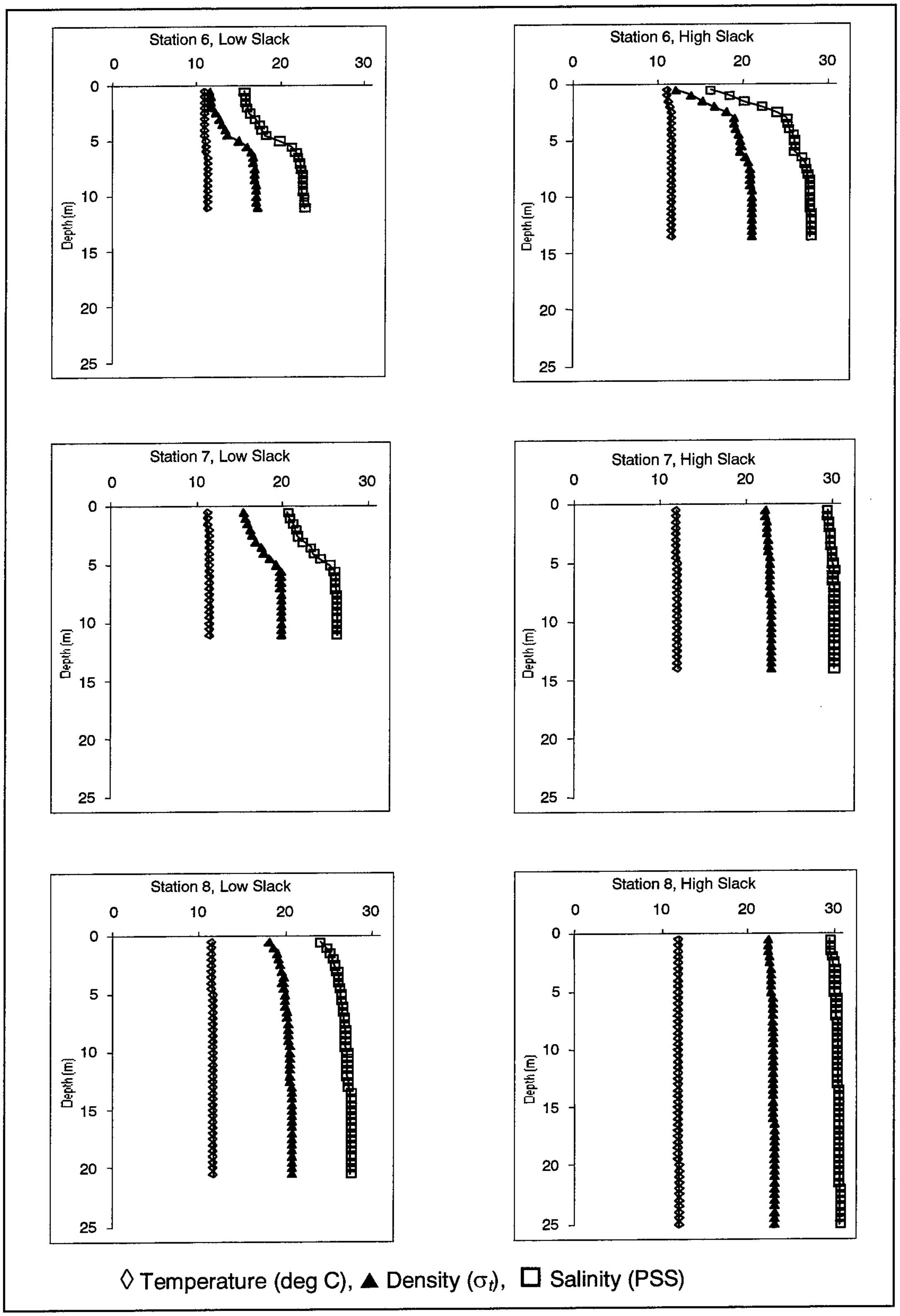

Figure 4-20. Temperature, density, and salinity profiles for stations in northern Willapa Bay during low- and high-slack tides (Stations 6-8)

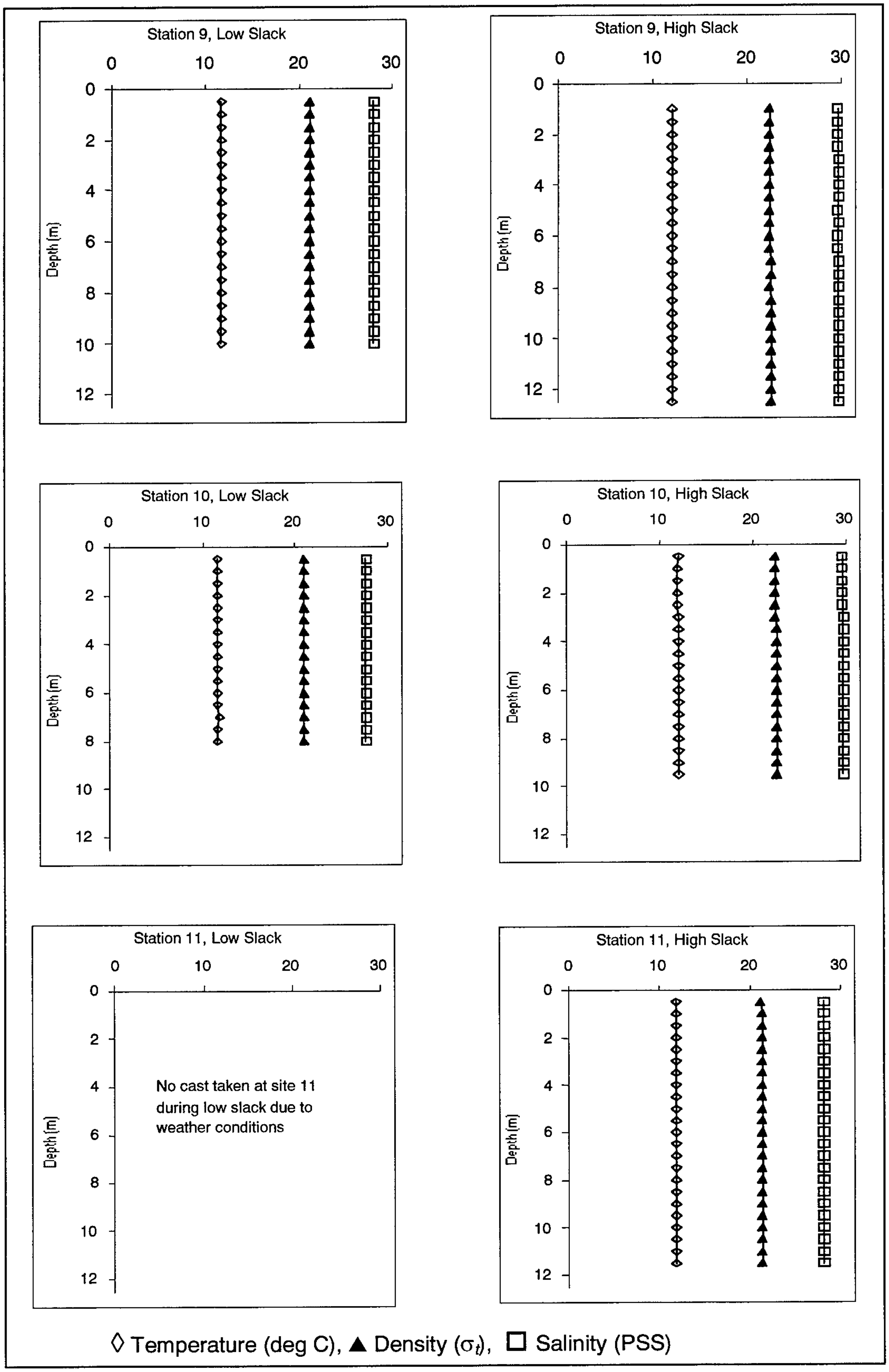

Figure 4-21. Temperature, density, and salinity profiles for stations in northern Willapa Bay during low- and high slack-tides (Stations 9-11)

Data trends

Wind speed averages remained steady until November, when a series of storms caused the average of that month to increase greatly over previous months (Table 4-11, Figures 4-22 to 4-26). Similarly, wind gusts did not vary greatly from August through November, but reached a maximum of 23.5 m/sec during the storms. Net wind direction was from the northwest during August and September (308 and 307 deg true north, respectively), then shifted to the southeast during October and November (158 and 160 deg true, respectively).

Air temperature decreased steadily throughout the study period with the maximum occurring in October (Table 4-11, Figure 4-24).

Average barometric pressure did not vary significantly between August and November. The lowest reading in November corresponded with a storm and its associated strong winds.

Table 4-11
Meteorological Data Summary

Parameter	Month	Average	Minimum	Maximum
Wind speed, m/sec	August	4.2	0.00	12.1
	September	3.9	0.01	11.3
	October	4.6	0.03	14.2
	November	6.6	0.60	17.0
Wind gusts, m/sec	August	5.4	0.0	14.7
	September	5.1	1.1	14.7
	October	6.0	1.1	17.9
	November	8.6	1.9	23.5
Air temperature, deg C	August	15.4	11.8	18.9
	September	13.8	10.1	20.4
	October	12.5	5.9	24.3
	November	10.3	6.7	13.3
Barometric pressure, mb	August	1,019.4	1,014.4	1,026.7
	September	1,015.1	1,008.7	1,022.4
	October	1,018.0	1,005.6	1,030.0
	November	1,014.2	1,000.6	1,021.9

North Channel Borrow Site Monitoring

PIE has been monitoring changes in the North Channel Dredged Borrow Site that was used for placing material on the shore near SR-105. Monitoring has included monthly hydrographic surveys starting after completion of the dredging work. Analysis of the data was performed through bathymetric data contouring, plotting cross sections of the survey area, and calculating sedimentation rates and volumes. Survey methodology, survey data processing, and analysis are presented in this report. The North Channel Borrow Site is shown in Figure 4-27. This area is referred to as the Borrow Site.

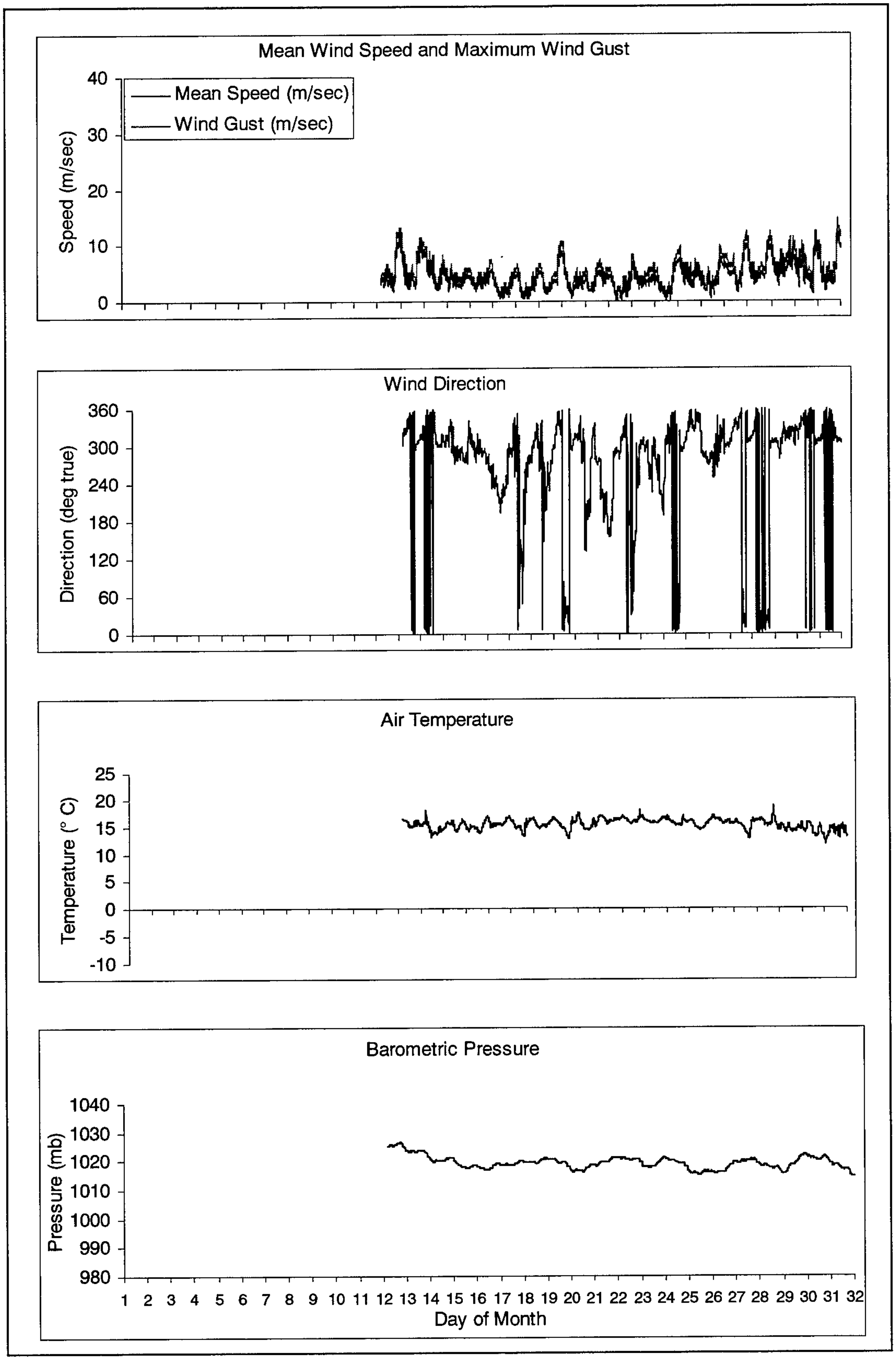

Figure 4-22. Meteorological data from central Willapa Bay, August 1998

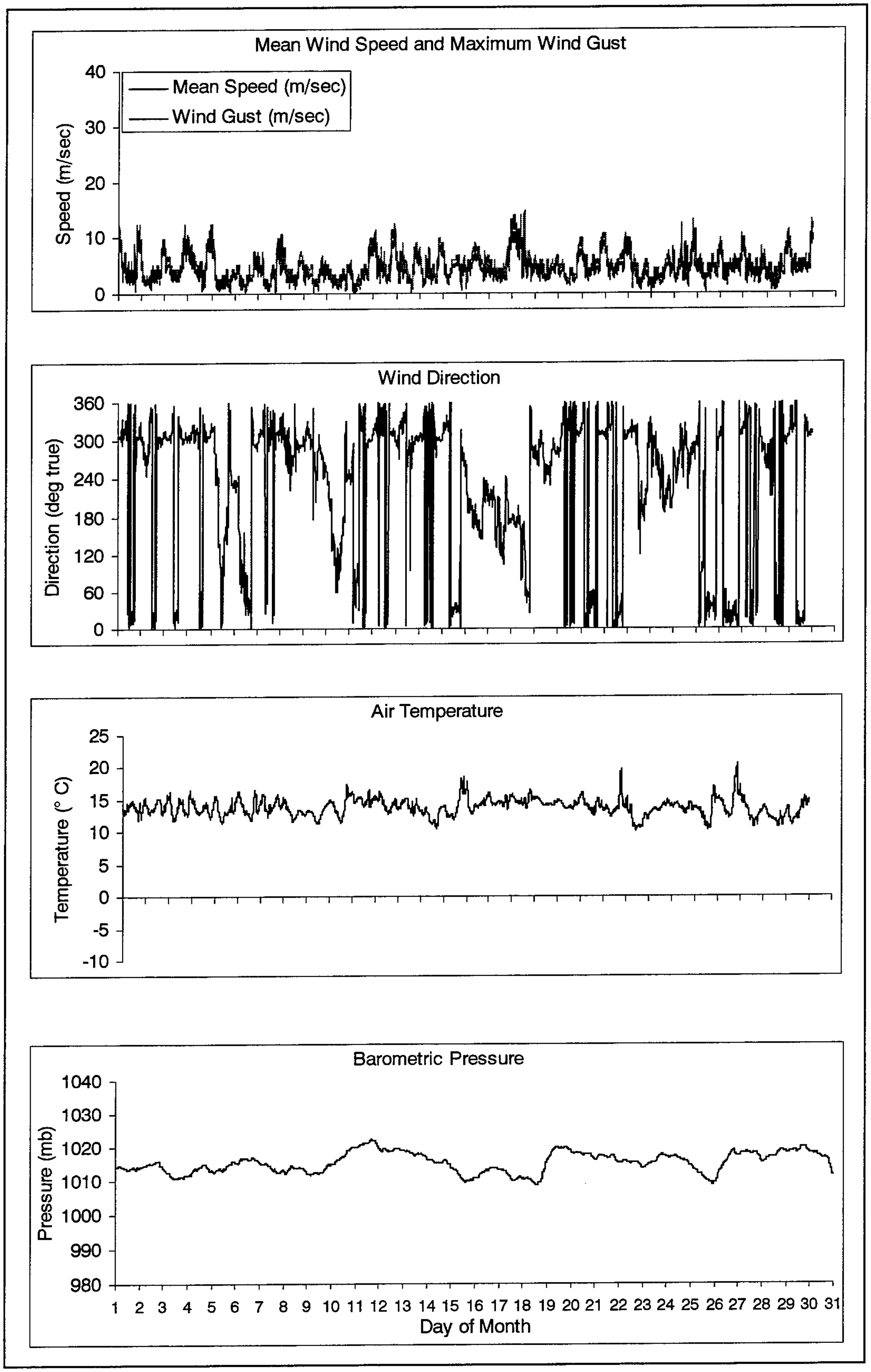

Figure 4-23. Meteorological data from central Willapa Bay, September 1998

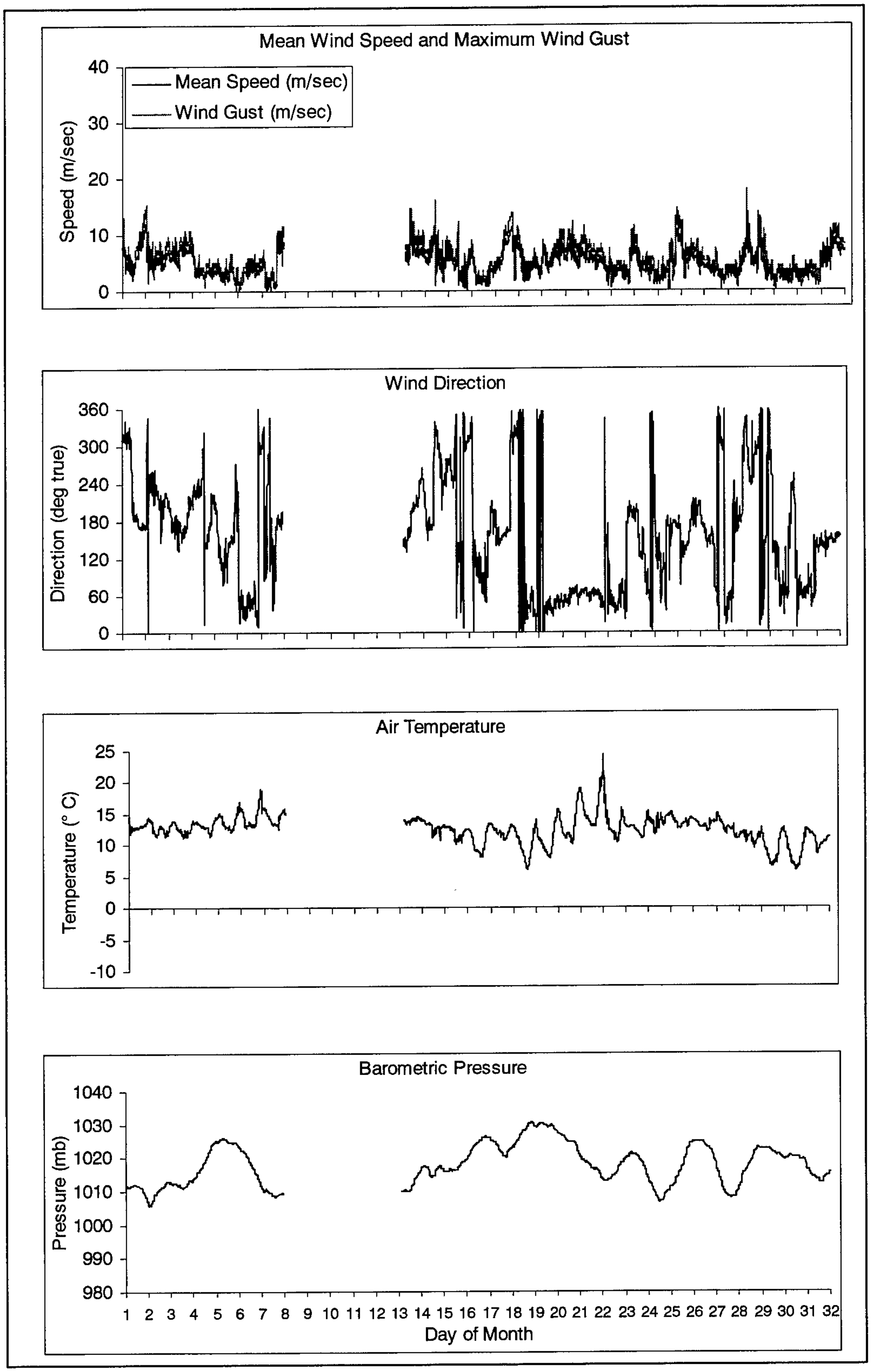

Figure 4-24. Meteorological data from central Willapa Bay, October 1998

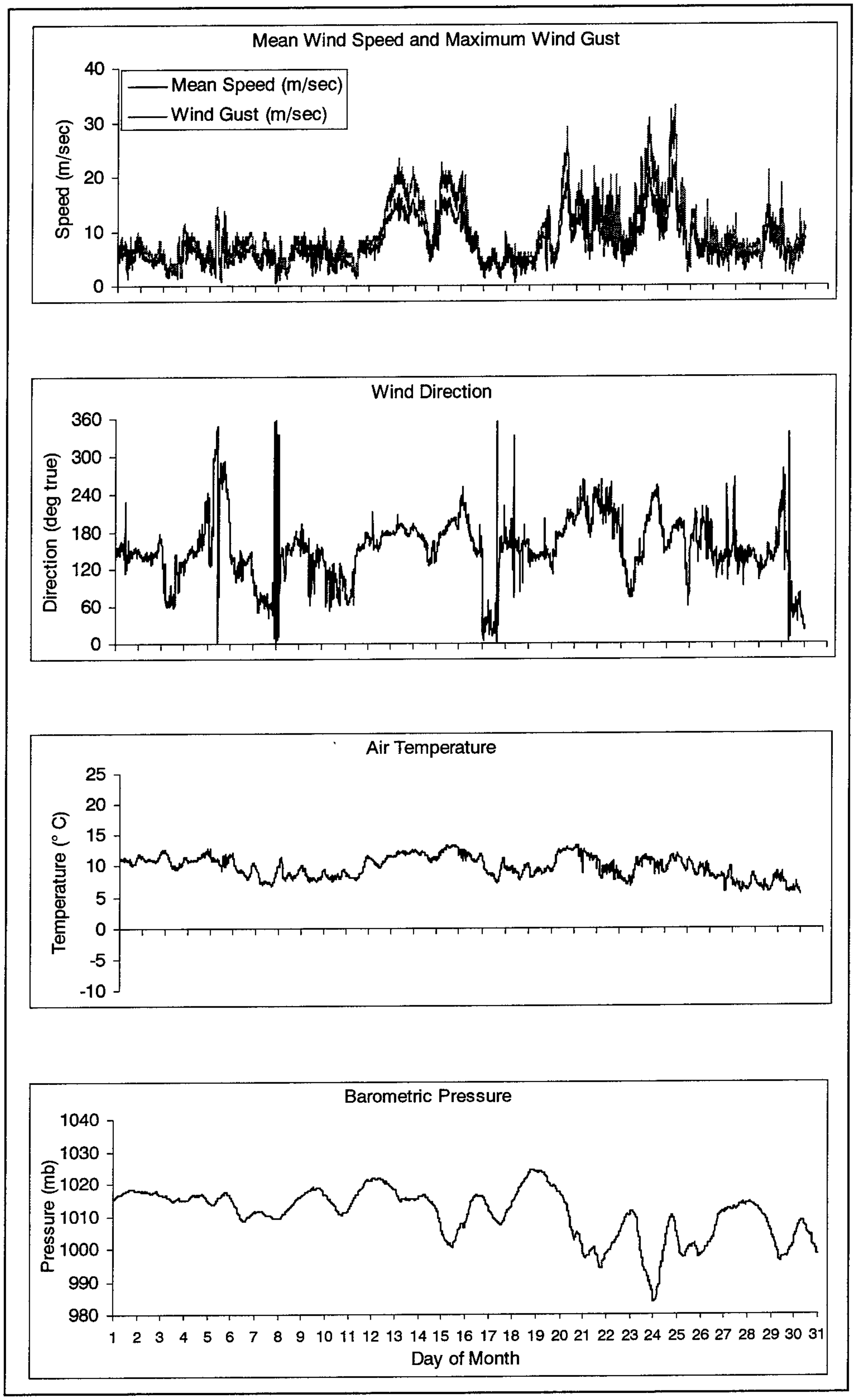

Figure 4-25. Meteorological data from central Willapa Bay, November 1998

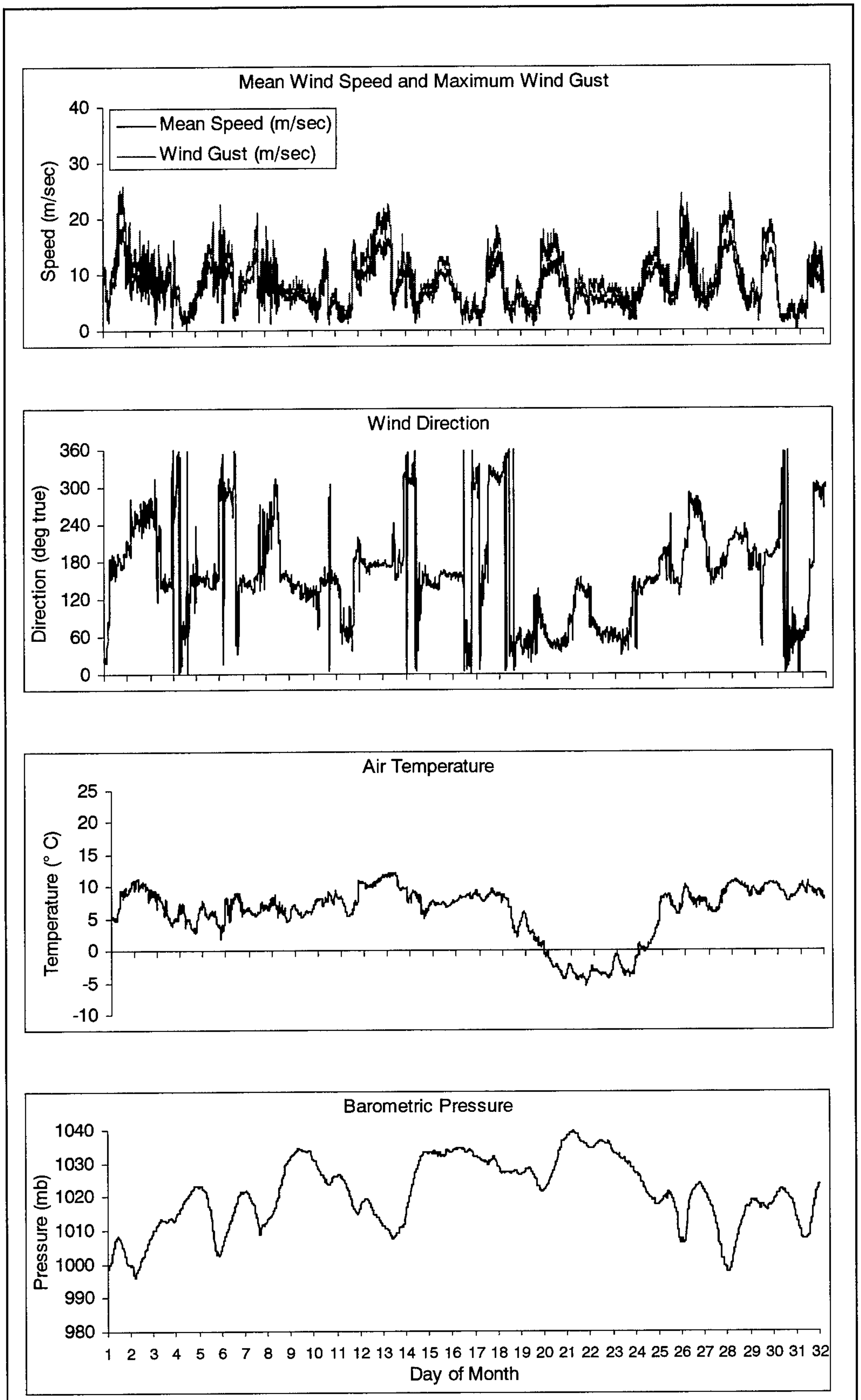

Figure 4-26. Meteorological data from central Willapa Bay, December 1998

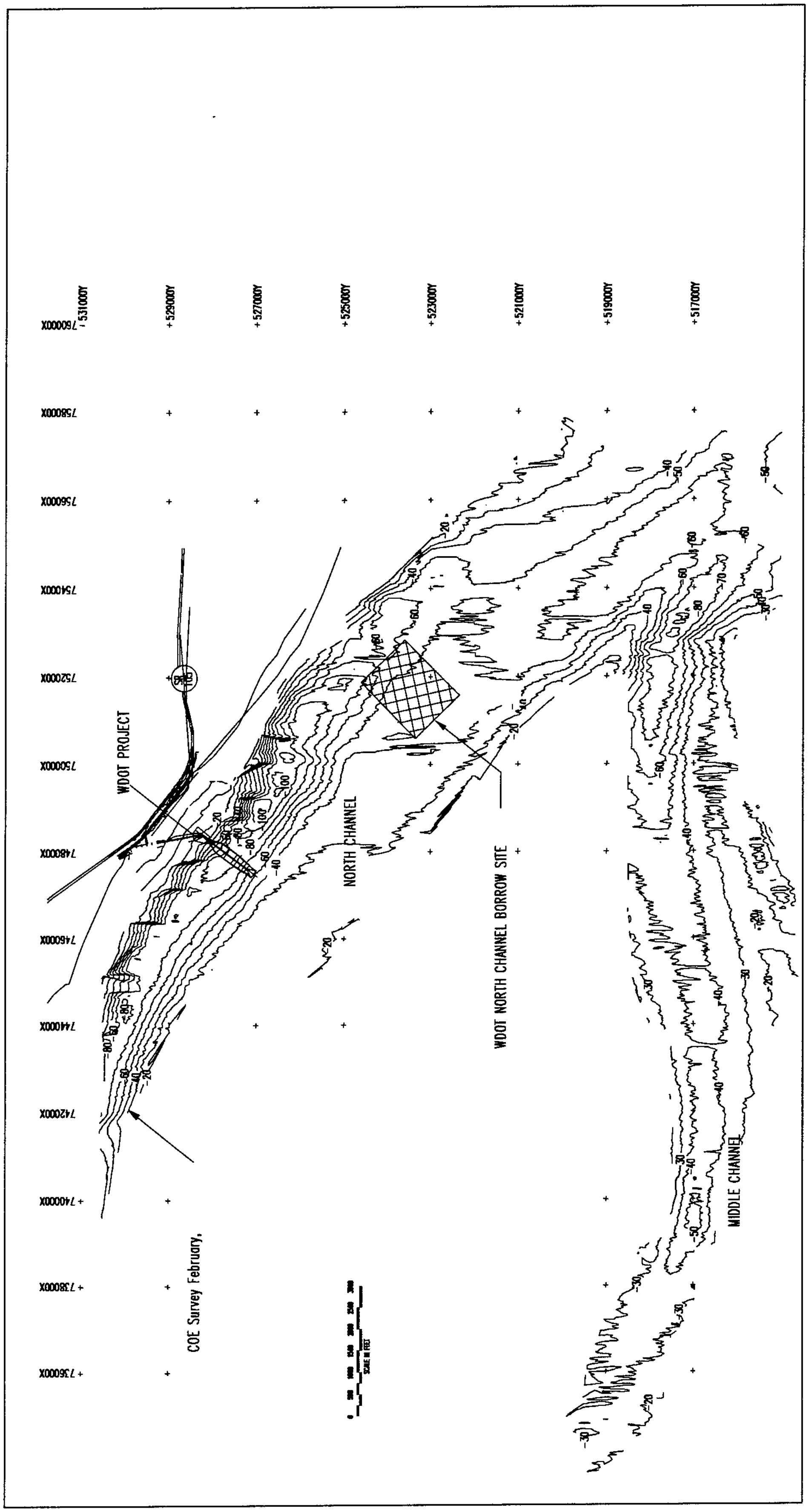

Figure 4-27. North Channel Borrow Site, Willapa Bay

Survey methodology

Bathymetric data were acquired with an Innerspace-448 precision survey echosounder (3-deg narrow-beam transducer) fixed to a 26-ft survey vessel. The echosounder and positioning equipment were interfaced to a computer-based data acquisition system running HYPACK (Coastal Oceanographics). A Trimble DSM-Pro DGPS receiver linked to the U.S. Coast Guard differential correction signal at Fort Stevens, Oregon, continuously provided position information. This positioning system provided continuous submeter horizontal position accuracy for all data collection. Soundings were corrected for vessel heave by an onboard heave/pitch/roll sensor (Seatex MRU-5).

Horizontal positions were logged as x- and y-coordinates in feet using the North American Datum of 1983 (NAD 83) and the Washington State Plane Zone #4602 (south). Depth values were logged in feet referenced to mllw and were corrected to the National Oceanic and Atmospheric Administration (NOAA) tide station at Toke Point using the integrated navigation software program HYPACK. Digitized depth soundings were checked against the analog record produced by the echosounder for additional quality control.

The WDOT Borrow Site survey area was established as a series of 65 parallel lines, oriented at 128 deg relative to true north, with 25-ft spacing between lines. This survey line configuration was established to be consistent with the predredging and postdredging surveys performed by David Evans and Associates, Inc., in June and July 1998. Surveys performed for the monthly monitoring of the Borrow Site were determined to require only a 100-ft line spacing. Therefore, every fourth line of the David Evans survey was run during each monthly monitoring survey. The horizontal extents of the survey are as follows:

a. North Corner: 751588 E, 524629 N.

b. West Corner: 752842 E, 523642 N.

c. South Corner: 751837 E, 522365 N.

d. East Corner: 750583 E, 523352 N.

Data quality was also verified by running survey lines oriented perpendicular to the primary survey lines ("tie lines"). All of the data acquisition systems, including the echosounder, heave/pitch/roll sensor, and the onboard computer systems, were calibrated before and after each hydrographic survey.

Hydrographic surveys of the Beach Nourishment Borrow Site (as of December 1998) were performed on 5 August, 4 September, 10 October, and 18 November 1998.

Survey data processing and analysis

Data obtained in XYZ format from each survey were used to generate a grid with 5-ft spacing between nodes. The grid was rotated clockwise 52 deg about point (0, 0) of the Washington State Plane NAD 83 horizontal datum. The rotation was performed consistently for every survey to create regularly shaped rectangular grids (software limitations). This manipulation was required because the North Channel Borrow Site is aligned with the North Channel bottom

contours. This rotation changed the northings and eastings of each bathymetric data point.

After grid generation, color contour maps were generated. Figure 4-28 shows the contoured survey data grids for surveys performed in June (predredging), July (postdredging), August, and September 1998. Figure 4-29 shows the contour maps for surveys performed in October and November 1998. The color bar scales in these figures give the corresponding elevations in the area surveyed relative to mllw. A comparison of the predredging and postdredging contour maps shows where dredging occurred. The Borrow Site is approximately 24 acres in size and was dredged to depths as large as 70 ft (mllw) in some areas. Figure 4-30 shows the depth changes that occurred between the postdredging survey in July and the PIE survey on 18 November 1998. Sedimentation of up to 20 ft was noticed in some areas of the Borrow Site.

Ten cross sections were taken through the surveyed area at 150-ft spacing. Figure 4-27 shows the location of each cross-section. Figure 4-31 shows cross sections 1, 2, 3 and 4. Each cross-section plot shows bottom surface variation obtained for the predredging and postdredging during the August, September, October, and November 1998 surveys. The dredge cut and subsequent deposition are clear on most of the cross sections. Figure 4-32 shows cross sections 5, 6, 7 and 8, Figure 4-33 shows cross sections 9 and 10.

Table 4-12 shows calculated changes in depth along each cross section. The average change in depth was calculated only within the identified dredged area and not along the whole cross section. Further averaging was performed to determine the average change in depth over the entire dredged area. For example, the Borrow Site was dredged about 7.7 ft on average over the entire site. Approximately 1 ft of accretion occurred between the postdredging survey taken in July and the 5 August 1998 survey. During the following month, erosion of 0.7 ft was noticed between 5 August and 4 September, followed by accretion of 1.8 ft between 4 September and 10 October. The most recent monitoring period showed accretion of 0.3 ft (between 10 October and 18 November 1998). It should be noted that erosion took place over the entire survey area between 5 August and 4 September.

Table 4-13 presents the volume changes calculated in the dredged hole between consecutive surveys. These volume changes were calculated using cross sections 1-10. Approximately 350,000 cu yd were dredged from the borrow site for beach nourishment. However, the volume change between predredging and postdredging surveys is about 315,000 cu yd in the dredged hole. This discrepancy may be attributable to filling of the dredged hole during the month-long dredging process. The dredged hole experienced accretion of approximately 42,000 cu yd from July to 5 August. Between 5 August and 4 September, approximately 24,000 cu yd of material was eroded from the dredge hole. Between 4 September and 10 October and between 10 October and 18 November, accretion of approximately 70,000 and 9,500 cu yd occurred, respectively.

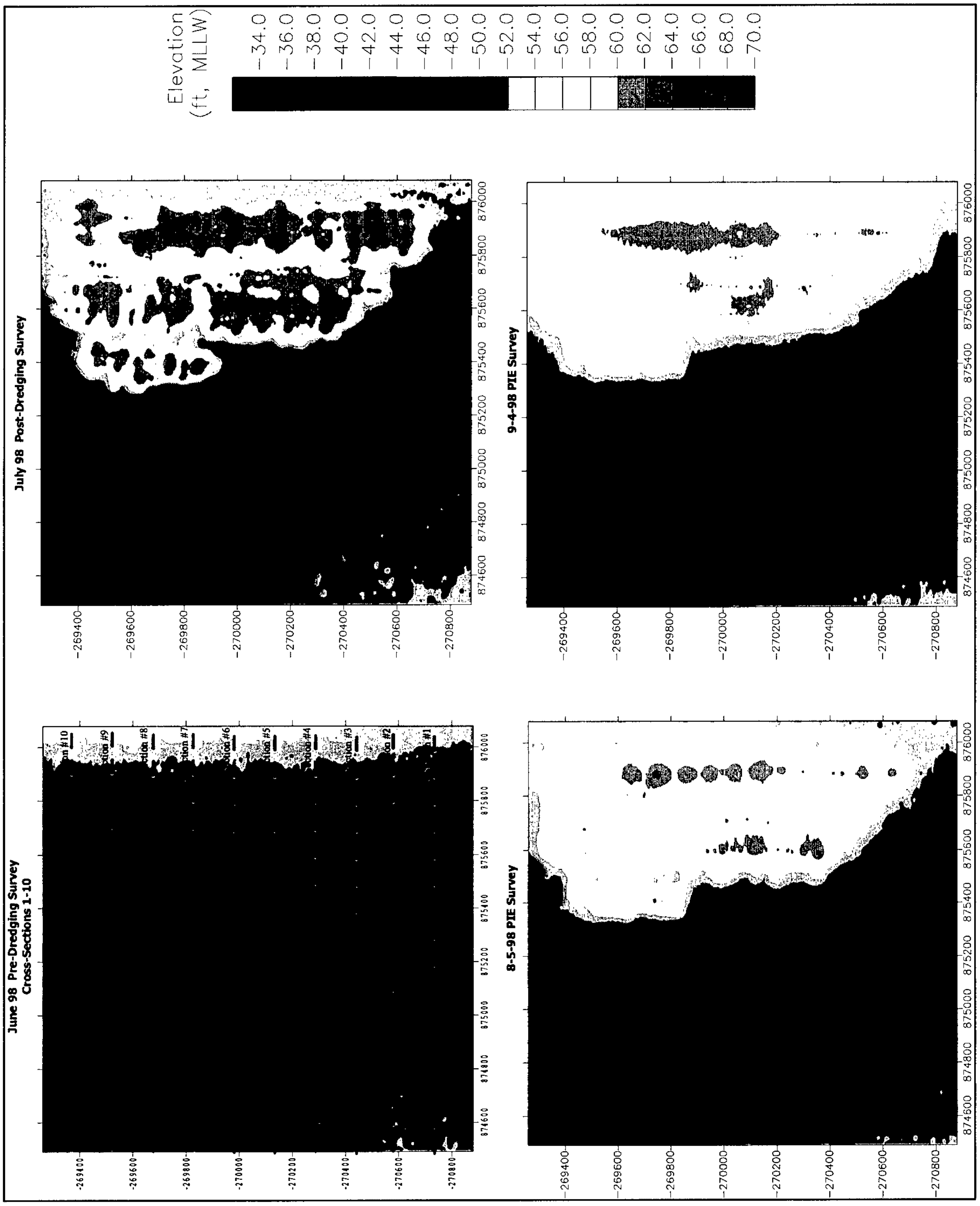

Figure 4-28. Contour maps of North Channel Borrow Site (predredging and postdredging, August and September 1998)

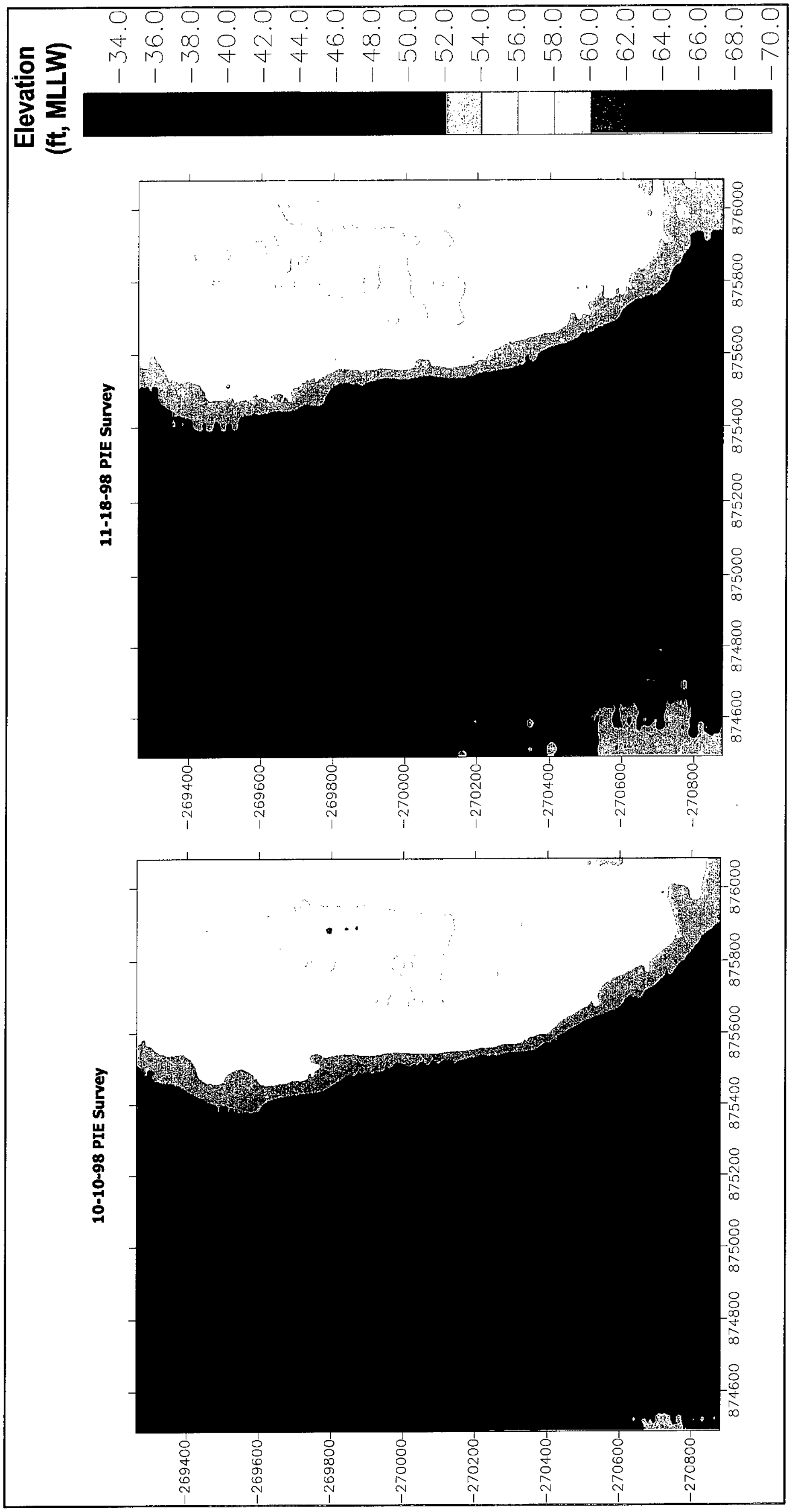

Figure 4-29. Contour maps of North Channel Borrow Site (October and November 1998)

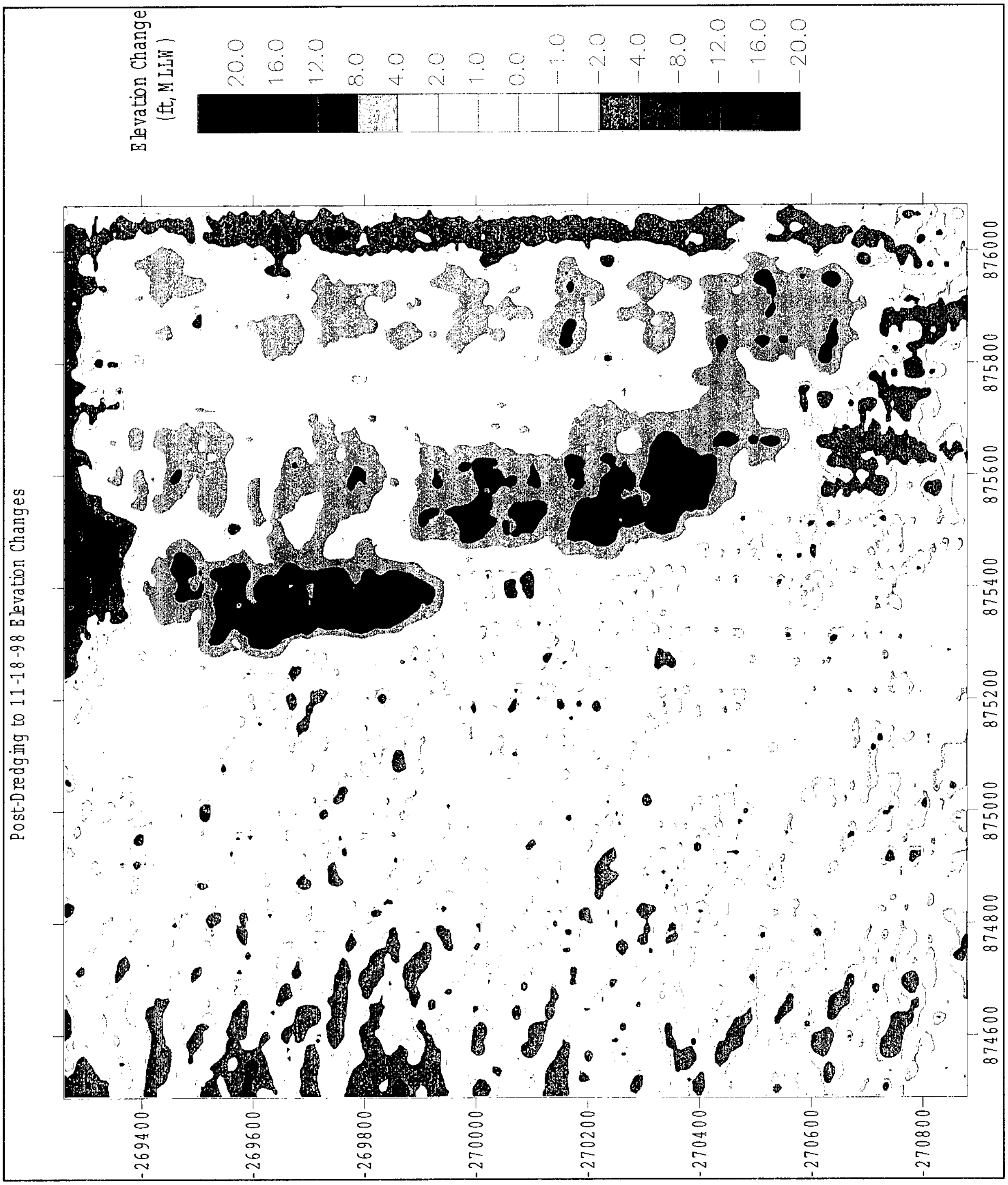

Figure 4-30. Contour maps of North Channel Borrow Site elevation changes (postdredging to November 1998)

Figure 4-31. Cross sections 1, 2, 3, and 4 of North Channel Borrow Site

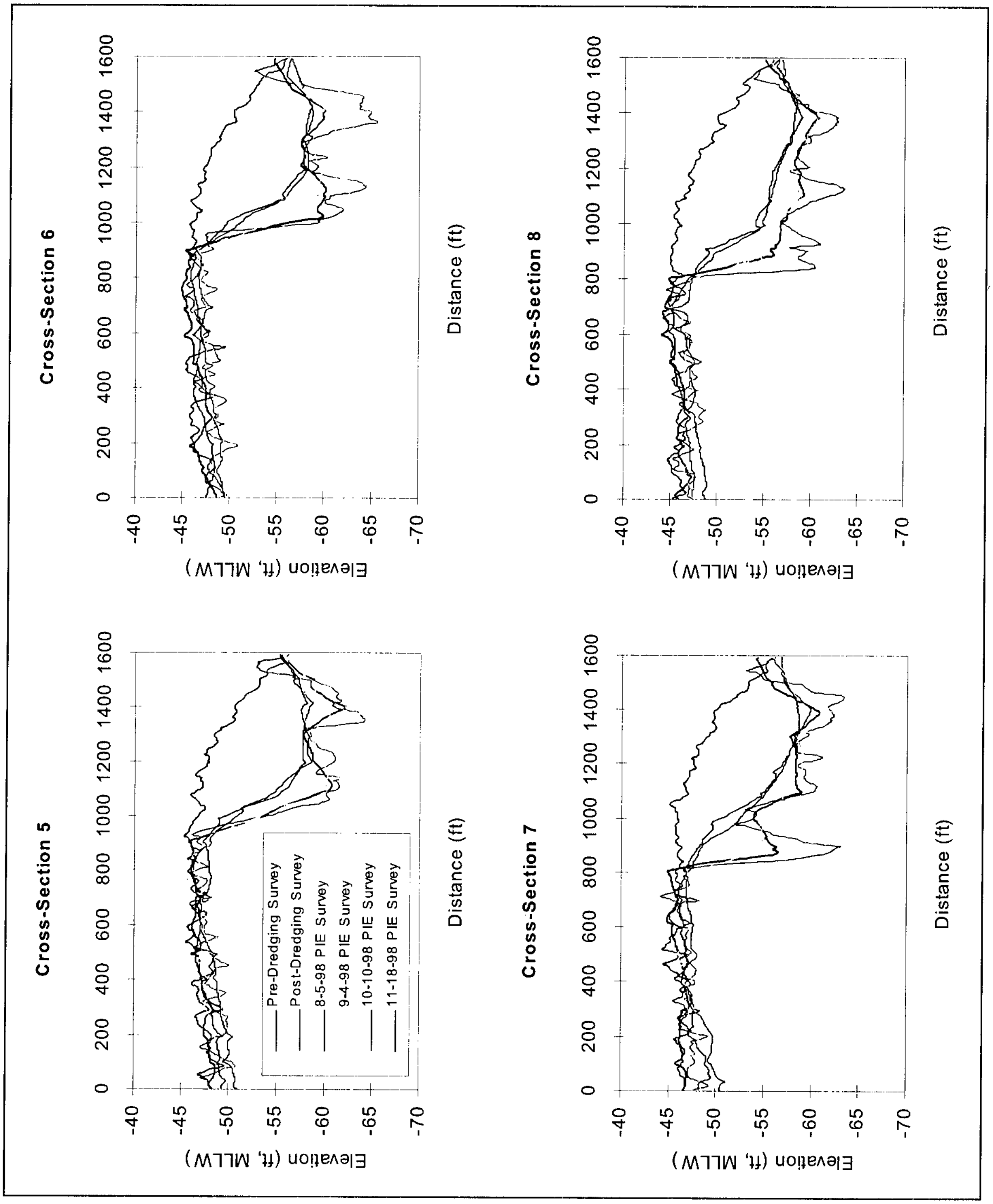

Figure 4-32. Cross sections 5, 6, 7, and 8 of North Channel Borrow Site

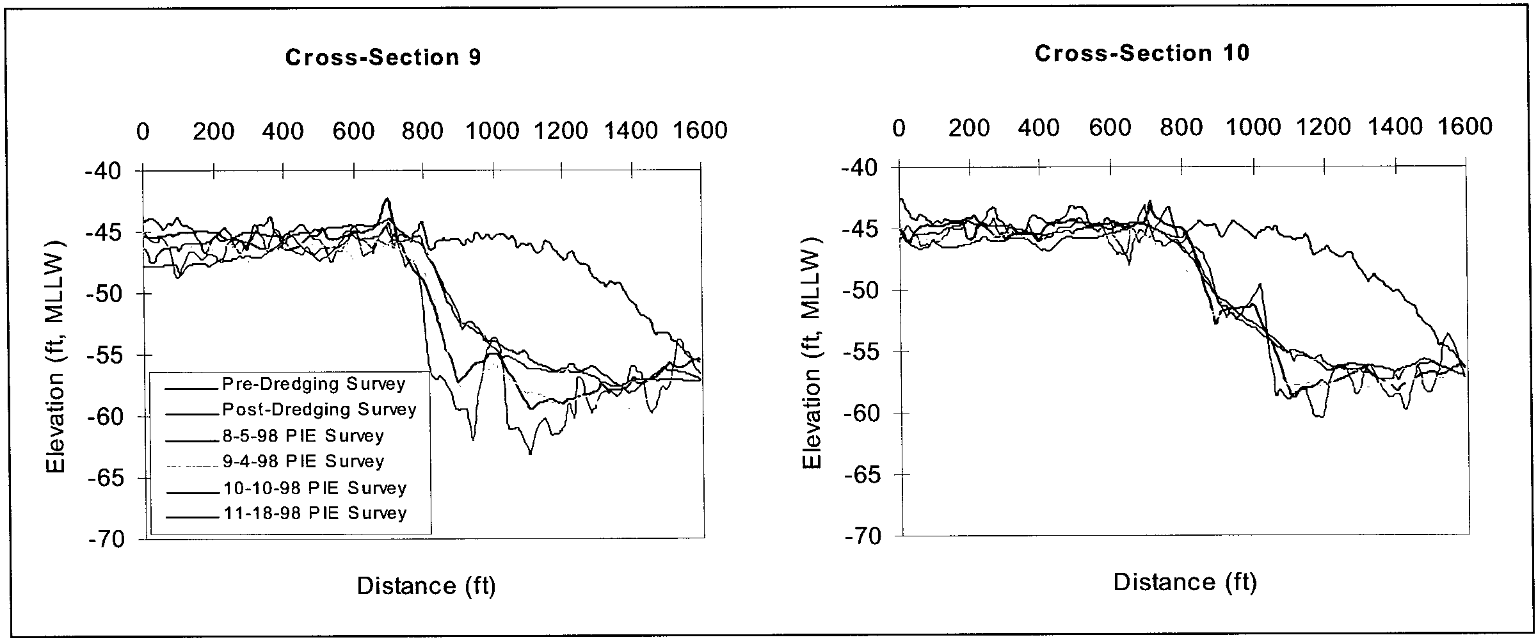

Figure 4-33. Cross sections 9 and 10 of North Channel Borrow Site

Table 4-12
Average Change in Depth Along Cross Sections, ft

Line Number	Pre to Post	Post to 8/5	8/5 to 9/4	9/4 to 10/10	10/10 to 11/18
1	-1.0	-0.5	-2.6	1.0	0.8
2	-5.6	0.7	-1.1	1.8	0.3
3	-9.0	2.1	-0.3	2.1	0.3
4	-8.0	0.9	-0.2	2.0	0.4
5	-8.7	0.5	-0.1	1.8	0.1
6	-9.7	1.7	-0.3	1.4	0.4
7	-9.3	1.6	-0.7	1.9	0.5
8	-10.2	1.5	-0.2	2.0	0.3
9	-9.0	1.4	-0.5	2.2	-0.2
10	-6.6	0.3	-1.0	1.5	-0.1
Average Over Dredged Area	-7.7	1.0	-0.7	1.8	0.3

Note:
Negative = Increase in depth (dredging or erosion).
Positive = Reduction in depth (accretion).

Table 4-13
Total Volume Changes (cu yd)

Pre to Post	Post to 8/5	8/5 to 9/4	9/4 to 10/10	10/10 to 11/18	Area of Borrow Site, sq ft
-312,482	41,969	-23,382	69,860	9,346	116,667

5 Modeling and Analysis of Short Waves[1]

Wind waves and swell that are generated by local or distant storms are defined as short waves. These surface gravity waves have periods less than about 25 sec. Quantitative information about short waves at the Willapa Bay entrance is required in this study for determining navigability and estimating sediment transport for evaluating navigation channel alternatives. Wave height, period, and direction (relative to the channel orientation) in part determine navigability. Sediment transport, which determines the frequency and cost of channel maintenance, is driven by a combination of short waves and currents as discussed in Chapter 6.

The purpose of this chapter is to describe modeling of the transformation of short waves across the Willapa bar and into the bay. First, the wave climate offshore of Willapa Bay is described. The offshore climate provides boundary conditions to initialize the wave model. Next, the STeady-state spectral WAVE transformation model STWAVE is described. STWAVE is based on the conservation of wave action, and it includes depth- and current-induced refraction, shoaling, and wave breaking. The bathymetry grid used in the model is presented next. Accurate bathymetry information is required, as discussed in Chapter 2, because waves refract and shoal according to the configuration of the sea bottom. The wave model is then evaluated with field measurements obtained in Chapter 4. Finally, simulations of the nearshore waves for evaluating channel design alternatives are presented.

Wave Climate

The first step in evaluating alternatives for the Willapa navigation channel is to define the wave climate seaward of the inlet, in relatively deep water. The offshore wave climate provides representative wave boundary conditions that initiate the wave transformation model. The model can then estimate the variation in wave height and direction along the channel. For highest accuracy, the source of wave information used to develop the offshore wave climate should be a site near Willapa Bay. Also, the water depth should be sufficient to avoid depth-limited breaking and refraction and shoaling induced by local nearshore contours or shoals. The source of wave data must have a long record (for

[1] Written by Dr. Jane McKee Smith and Mr. Bruce A. Ebersole, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS.

accurate climate statistics) and be presently operating (to facilitate verification with 1998 data collected at Willapa).

Offshore wave information near Willapa Bay is available from four sources:

a. Directional wave buoy offshore of Grays Harbor, Washington. This is a Datawell buoy located at 46°51.4′N, 124°14.7′W, in a depth of 40.2 m (Station 03601). The buoy is supported by U.S. Army Engineer District, Seattle, through the Corps of Engineers Field Wave Gauging Program and operated by the Scripps Institute of Oceanography (SIO), Coastal Data Information Program (CDIP). The buoy was installed in 1993 and is still operating.

b. Bottom-mounted slope array offshore of Long Beach, Washington. The slope array was located at 46°23.2′N, 124°4.7′W, in a depth of 10 m (Station 05401). The gauge was supported by the Corps of Engineers Field Wave Gauging Program and operated by the SIO CDIP. The gauge was installed in 1983 and operated until mid-1995.

c. Directional wave buoy offshore of the Columbia River Bar, Oregon. This is a 3-m discus buoy located at 46 7′N, 124°30′W, in a depth of 128 m (Station 46029). The buoy is supported and operated by the National Data Buoy Center. The buoy was installed in 1984 and is still operating.

d. The U.S. Army Corps of Engineers' Wave Information Study (WIS) wave hindcast. The closest WIS Phase II station is located at 46°16.2′N, 124°46.2′W, in deep water (Station 46) (Corson et al. 1987). The WIS hindcast covers the period 1956-1975.

The Grays Harbor buoy and the Long Beach gauge are located closest to Willapa Bay. The Grays Harbor buoy is approximately 20 km northwest of the Willapa entrance, and the Long Beach gauge was about 30 km south of the entrance. Because the Long Beach gauge was in relatively shallow water, wave direction was strongly influenced by the local bathymetry, and the higher waves would break at the site, biasing the distribution of the highest wave heights. Also, the Long Beach gauge is no longer operational, so evaluation of the wave model with wave data collected at the inlet in 1998 cannot use the Long Beach gauge for incident wave conditions. The Columbia River Bar buoy and the WIS information provide longer periods of records than the Grays Harbor buoy, but are also much further away from Willapa Bay. Also, the WIS information does not overlap with the 1998 Willapa Bay measurements for evaluation of the model. For these reasons, the Grays Harbor buoy was selected to determine the wave climate for Willapa Bay. Appendix E contains comparisons of wave climate statistics from the Grays Harbor buoy, the Long Beach slope array, and the Columbia River buoy.

A wave climate was developed using Grays Harbor data from September 1996 through August 1998. In September 1996, the reporting of wave angles changed from an arithmetic weighted average to a peak wave direction (wave direction band with the maximum energy). Thus, for consistency, only the newer data were used. Although a 2-year data record is short for describing extreme wave statistics, it is sufficient to characterize waves for navigation and typical storms to assess channel shoaling. To construct the wave climate, percent

occurrence tables (broken down by height, period, and direction) were calculated for each month of data and then the monthly tables were combined, giving equal weight to each month. The equal monthly weighting takes into account gaps in the data and changes in sampling frequency. The Grays Harbor wave climate is illustrated in Figure 5-1 as a wave rose with directional resolution of 22.5 deg. Table 5-1 shows overall distributions by height, period, and direction.

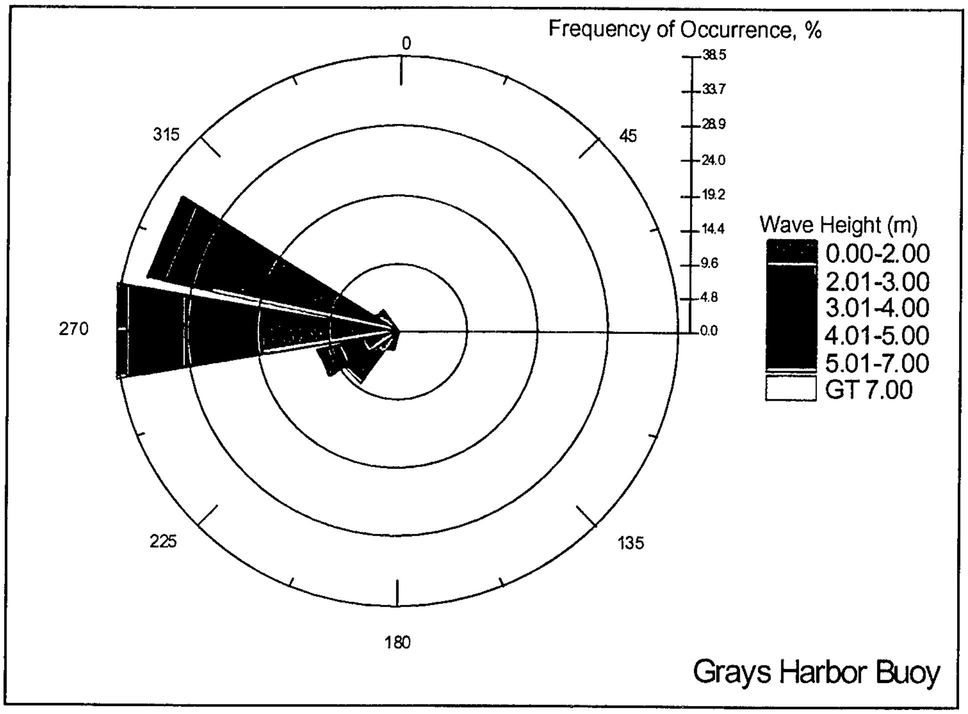

Figure 5-1. Grays Harbor buoy wave rose

Table 5-1
Grays Harbor Wave Distribution by Height, Period, and Direction

Height m	Occurrence percent	Period sec	Occurrence percent	Direction deg	Occurrence percent
0.00-2.0	57.5	0.00- 6.0	2.8	180.0	0.4
2.01-3.0	24.7	6.01-10.0	52.2	202.5	2.7
3.01-4.0	11.1	10.01-14.0	28.1	225.0	8.7
4.01-5.0	4.4	14.01-18.0	13.8	247.5	11.2
5.01-7.0	2.2	>18.01	3.1	270.0	38.5
>7.01	0.1			292.5	34.6
				315.0	3.7
				337.5	0.1

The average conditions over the period August 1993 through July 1999 were wave height of 6.9 ft, peak period of 10.6 sec, and peak direction of 273 deg (based on wave data revised by CDIP in the spring of 1999). A monthly distribution of the mean and maximum wave heights, periods, and directions and occurrence of heights less than 9 ft are given in Table 5-2 (maximum period and direction given in the table are the values associated with the maximum wave height). The maximum wave height in the record is 29.6 ft, with a period of 13.3 sec and direction of 234 deg (24 November 1998 at 0719 Greenwich mean time (GMT)). Large winter storms impede navigation at the Willapa entrance channel. Based on the Grays Harbor buoy, wave height exceeds 9.8 ft 42 percent of the time and exceeds 4.9 ft 92 percent of the time during winter months (December-February). Time-histories of winter wave heights and directions from the Grays Harbor buoy are given in Appendix C.

Table 5-2
Mean and Maximum Wave Parameters Measured at Grays Harbor Buoy (August 1993 - July 1999)

Month	Mean Height ft	Mean Period sec	Mean Direction deg	Occurrence of $H \leq 9$ ft percent	Maximum Height ft	Maximum Period[1] sec	Maximum Direction[1] deg
January	8.83	11.6	257	57	27.8	13.3	258
February	9.84	13.0	261	48	24.3	13.3	269
March	8.14	12.0	264	63	23.1	12.5	238
April	6.92	11.0	274	79	18.2	11.1	290
May	5.54	9.4	278	95	13.3	11.8	278
June	5.15	9.4	283	95	11.6	15.4	261
July	4.27	8.1	287	99	11.3	10.0	300
August	3.84	8.5	283	100	9.8	9.1	283
September	5.28	9.6	282	95	12.2	11.8	281
October	6.92	10.9	275	78	21.3	12.5	269
November	9.12	11.1	264	58	29.6	13.3	234
December	10.50	12.4	264	36	25.7	14.3	272

[1]Value associated with the maximum wave height.

Wave Transformation Model

The numerical model STWAVE (Resio 1987, 1988; Smith, Resio, and Zundel 1999) was used to transform waves across the Willapa bar for evaluation of channel alternatives. STWAVE numerically solves the steady-state conservation of spectral action balance along backward-traced wave rays:

$$(C_{ga})_x \frac{\partial}{\partial x} \frac{C_a C_{ga} \cos(\mu - \alpha) E(f,\alpha)}{\omega_r} + (C_{ga})_y \frac{\partial}{\partial y} \frac{C_a C_{ga} \cos(\mu - \alpha) E(f,\alpha)}{\omega_r} = \sum \frac{S}{\omega_r} \tag{5-1}$$

where

C_{ga} = absolute wave group celerity

x,y = spatial coordinates; subscripts indicate x-and y-components

C_a = absolute wave celerity

μ = current direction

α = propagation direction of spectral component

E = spectral energy density

F = frequency of spectral component

ω_r = relative angular frequency (frequency relative to the current)

S = energy source/sink terms

The source terms include wind input, nonlinear wave-wave interactions, dissipation within the wave field, and surf-zone breaking. The terms on the left-hand side of Equation 5-1 represent wave propagation (refraction and shoaling), and the source terms on the right-hand side of the equation represent energy growth or decay in the spectrum.

The assumptions made in STWAVE are as follows:

a. Mild bottom slope and negligible wave reflection.

b. Spatially homogeneous offshore wave conditions.

c. Steady waves, currents, and winds.

d. Linear refraction and shoaling.

e. Depth-uniform current.

f. Negligible bottom friction.

STWAVE is a half-plane model, meaning that only waves propagating toward the coast are represented. Waves reflected from the coast or waves generated by winds blowing offshore are neglected. Wave breaking in the surf zone limits the maximum wave height based on the local water depth and wave steepness:

$$H_{mo_{\max}} = 0.1L \tanh kd \tag{5-2}$$

where

H_{mo} = zero-moment wave height

L = wavelength

k = wave number

d = water depth

STWAVE is a finite-difference model and calculates wave spectra on a rectangular grid with square grid cells. The model outputs zero-moment wave

height, peak wave period T_p, and mean wave direction α_m at all grid points and two-dimensional spectra at selected grid points.

Wave Model Inputs

The inputs required to execute STWAVE are as follows:

a. Bathymetry grid (including shoreline position and grid size and resolution).

b. Incident frequency-direction wave spectrum on the offshore grid boundary.

c. Current field (optional).

d. Tide elevation, wind speed, and wind direction (optional).

Bathymetry grid

Both the wave and circulation models (Chapter 6) require bathymetry data to construct computational grids over which the waves propagate and transform. Accurate bathymetry is required for modeling waves at Willapa Bay because the complex shoals control transformation and breaking. Bathymetry data collected in this study include high-resolution Lidar surveys with Scanning Hydrographic Operational Airborne Lidar Survey (SHOALS) (Lillycrop, Parson, and Irish 1996) and broad-coverage surveys collected by the Seattle District survey boat, the *Shoalhunter*. These data were combined with existing National Ocean Service (NOS) data to provide coverage of the nearshore, bar, and bay (Chapter 3).

Very shallow areas across the Willapa bar could not be surveyed because of continuous wave breaking. Bathymetry in these areas was estimated from local knowledge.[1,2] A bathymetry grid for the circulation model was generated from the combined SHOALS, *Shoalhunter*, and NOS data to represent the existing conditions at Willapa (see Chapter 6). For wave modeling, the unstructured circulation model grid was linearly interpolated onto a rectilinear STWAVE grid. The STWAVE grid has a resolution of 100 m, with 301 grid cells across the shore and 511 cells along the shore. The area of coverage is 30 km (west to east) by 51 km (south to north). The grid origin (southwest corner of the grid) is located at the Universal Transverse Mercator (UTM) Northing 5,136,069 m (lat. 46.3719°N) and Easting 407,546 m (long. 124.2021°W). The existing condition bathymetry and STWAVE grid coverage are shown in Figure 5-2. Depths are given in meters relative to the mean tide level (mtl). Note that other chapters typically reference depths to mean lower low water (mllw) datum, and a value of 1.52 m mllw was used to convert depths to mtl based on the tidal datum at Toke Point. In addition to the grid generated for existing bathymetry, grids were developed in the same manner for Alternatives 3A, 3B, 4A, and 3H. Each of these grids incorporates modifications to the existing bathymetry that correspond to the particular alternative.

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[1] Personal Communication, October 1998, Mr. Thomas Landreth, *Shoalhunter* Captain, U.S. Army Engineer District Seattle, Seattle, WA.

[2] Personal Communication, October 1998, Mr. Randy D. Lewis, City Administrator, Westport, WA.

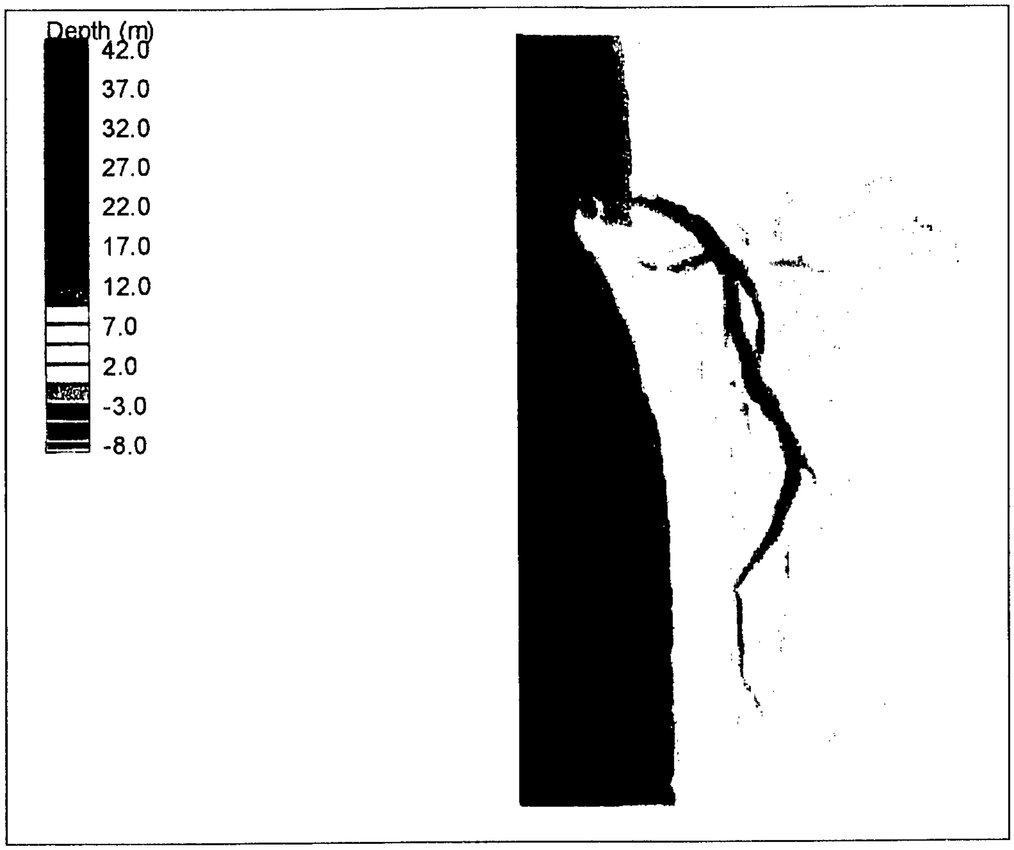

Figure 5-2. STWAVE existing condition bathymetry grid

Input wave spectra

Input wave spectra are the forcing for the wave model and provide the distribution of wave energy as a function of frequency and direction. For this project, the input spectra were generated in two ways:

a. Verification spectra. For evaluation of the wave model with field measurements (1-4 September 1998, 15-20 October 1998, and 11-16 November 1998), one-dimensional frequency spectra from the Grays Harbor buoy were used. The directional distributions measured at the buoy lack sufficient resolution to drive the model, so a theoretical distribution of the form $\cos^{nn}(\alpha - \alpha_m)$ was applied, where *nn* is the spreading coefficient, and α_m is the mean wave direction. The values used for *nn* are given in Table 5-3 (Thompson et al. 1996). Large values of *nn* indicate a narrow directional distribution (swell waves), and small values represent a wide distribution (sea waves). The value of *nn* for the peak of the spectrum (Table 5-3) was applied for the peak and lower frequencies. For frequencies higher than the peak, the values in Table 5-3 were applied.

Table 5-3
Values of *nn* Defining Direction Distribution in Verification Spectra

f, Hz	≤ 0.04	0.045	0.05	0.055	0.06	0.08	≥ 0.1
nn	38	36	30	26	22	10	4

b. Representative spectra. For model runs made to evaluate channel alternatives and sediment-transport potential, representative wave spectra were generated. These spectra are representative of the height, period, and direction ranges shown in Table 5-1. The spectra were generated with a TMA[1] frequency distribution (Bouws et al. 1985) and a $\cos^{nn}(\alpha - \alpha_m)$ directional distribution. The TMA shape is defined by the wave height, peak period, and spectral peakedness parameter γ. Large values of γ indicate a narrow frequency distribution (swell waves), and small values represent a wide distribution (sea waves). Combinations of γ and *nn* used to generate the representative spectra are given as a function of peak period in Table 5-4.

Table 5-4
Values of and *nn* Defining Representative Spectra

T_p (sec)	5	8	12	16	20
γ	3.3	3.3	4	6	8
nn	4	4	10	20	30

Current fields

For applications where wave-current interaction significantly alters the wave height or blocks the waves, current fields are needed as an input to the model. Wave height increases on strong ebb currents and decreases on strong flood currents. Currents also alter wave direction. The current modifies waves with higher frequencies (shorter periods) more than it does waves with lower frequencies. In addition to shoaling and refraction by currents, wave breaking is also changed by an opposing or following current (L and k change in Equation 5-2). Wave breaking is enhanced on an opposing current (ebb) and reduced on a following current (flood). If the ebb current is strong, waves with short periods cannot propagate against it, and wave energy is blocked and dissipated.

For Willapa, model runs with and without a current were made to assess the sensitivity of the wave transformation at the study site to wave-current interaction. For the sensitivity analysis, relatively commonly occurring short- and long-period waves at Willapa Bay were selected: $H_{mo} = 1.5$ m, $T_p = 8$ sec, and $\alpha_m = 292.5$ deg (probability of occurrence of 21.6 percent) and $H_{mo} = 2.5$ m,

[1] Named for the three data sets used to develop the spectrum: TEXEL storm, MARSEN, and ARSLOE.

T_p = 16 sec, and α_m = 270 deg (probability of occurrence of 2.9 percent). The waves were run for a typical peak ebb and flood, and the first wave was also run for a maximum spring tide ebb and flood.

Figures 5-3 and 5-4 are plots of the difference in wave height with and without current for the first wave and typical ebb and flood conditions, respectively (Cases 1 and 2 in Table 5-5). The increase in wave height on the outer bar for a typical ebb is about 2 ft (0.6 m), which is equal to the value estimated based on local knowledge.[1] Table 5-5 summarizes the maximum differences in wave height (height modified by the current minus height neglecting current) for ebb and flood currents. The percent differences are calculated as the difference in wave height calculated with and without currents divided by the height calculated without currents.

Table 5-5
Sensitivity Analysis of Wave Transformation to Current

Case	Incident Wave Conditions H_{mo} m	T_p sec	α_m deg	Max North Channel Current m/sec	Max Wave Height Difference m	Max Wave Height Difference percent
1	1.5	8	292.5	1.6 ebb	0.6	67
2	1.5	8	292.5	1.4 flood	-0.2	-15
3	2.5	16	270	1.6 ebb	0.4	17
4	2.5	16	270	1.4 flood	-0.4	-9
5	1.5	8	292.5	2.0 ebb	0.8	79
6	1.5	8	292.5	1.6 flood	-0.2	-16

[1] Personal Communication, October 1998, Mr. Randy D. Lewis, City Administrator, Westport, Washington. Mr. Lewis has served in the U.S. Coast Guard and is familiar with the channels and sea state at the Willapa entrance.

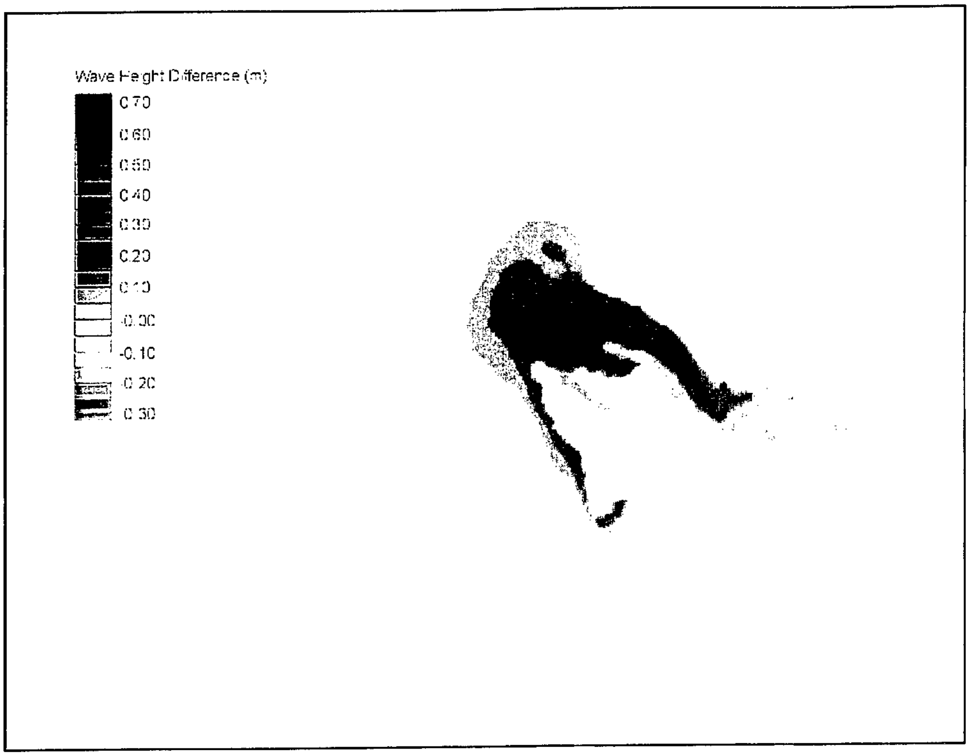

Figure 5-3. Wave height with ebb current minus wave height with no current

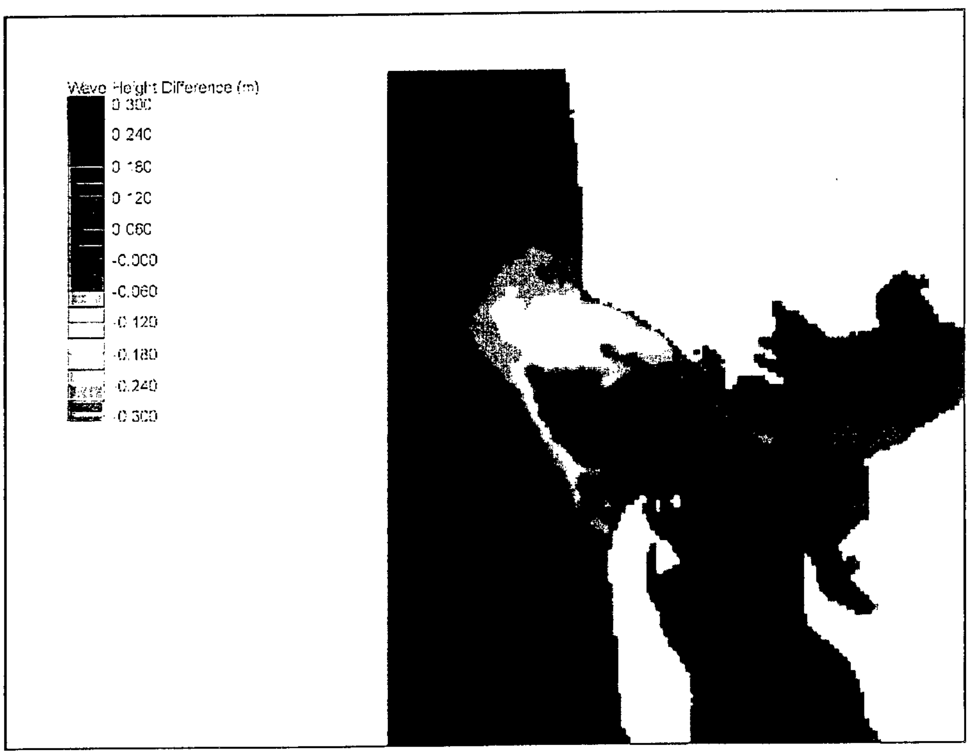

Figure 5-4. Wave height with flood current minus wave height with no current

The maximum differences in wave height with and without current are significant (0.4 to 0.8 m on ebb and 0.2 to 0.4 m on flood). The largest differences occur in the North Channel through the bar, so by the present sensitivity analysis, wave-current interaction should be included in evaluation of navigability. However, the model shows that the regions where wave-current interaction is significant are fairly small. Regions of wave-height differences greater than 30 percent (peak ebb) were seven grid cells wide (700 m) or less (which is less than 9 percent of the inlet width). Thus, wave-current interaction need not be considered in estimating channel shoaling, which is determined more by wave breaking, wave-driven currents, and sediment transport on the ebb shoals than by wave height in the navigation channel. Wave-current interaction was included for wave model runs to assess navigability, but this intensive, iterative calculation was neglected for runs performed to provide sediment transport forcing.

Tide and wind

Tide elevation is applied in STWAVE as constant water depth change over the entire grid. Within the Willapa grid domain, the tide elevation does vary spatially, but the influence of this variation on wave transformation is relatively small (and is on the order of wind and wave setup, which are neglected). Tide elevation for wave runs is specified from either tide measurements at the NOS Toke Point station or, for typical conditions, set to mtl (0 m), mean low water (mlw) (-1 m), or mean high water (mhw) (+1 m). Because the grid depths are specified relative to mtl, tide fluctuations are specified relative to mtl.

Wind input in STWAVE creates wave growth across the grid domain. Wave measurements at the Grays Harbor buoy contain most of the local wave-generation processes. Fetch lengths from the buoy depth (40 m) to the Willapa entrance are short, so additional growth would be small. Thus, local generation is neglected in the STWAVE calculations, including locally generated waves within Willapa Bay.

Evaluation of Model with Field Data

Willapa is a challenging environment in which to apply a wave transformation model. The inlet is subjected to high waves, strong currents, and large variations in water elevation; and the bathymetry at the Willapa entrance is complex and continually changing. Before STWAVE was applied to simulate project alternatives, the model was evaluated with field data to assess the accuracy for such demanding conditions. The initial emphasis of the project was the Middle Channel; thus, wave gauges were deployed at Middle Channel Stations 1, 2, and 3 (see Chapter 4 for discussion of field measurements) to provide verification data. Three verification periods were selected for modeling: 1-4 September 1998, 15-20 October 1998, and 11-16 November 1998. These periods include wave heights of 1.0 to 5.5 m, periods of 4 to 18 sec, offshore wave directions of 205 to 306 deg, peak tidal amplitudes of 1 to 1.5 m, and peak near-bottom currents of 0.5 to 1.2 m/sec. These wave conditions are representative of waves in which vessels navigate through the entrance, as well as the most commonly occurring waves. Measurement accuracy is estimated as

about ±10 percent for wave height and ±10 deg for direction. An evaluation of the pressure gauge record used for the wave measurements is given in Appendix F. The root-mean-square (rms) error in wave height for the pressure gauge, compared with that of a wave buoy, is approximately 0.1 m. The pressure gauge generally underestimates small wave heights (at low tide) and is unbiased at other tide elevations.

Wave transformation at Willapa is sensitive to the minimum water depth across the Willapa bar. The initial attempt to verify STWAVE for the existing Willapa bathymetry produced poor agreement with the measurements. STWAVE gave little wave attenuation between Station 1 and Stations 2 and 3. The lack of agreement called into question the bathymetry data across the Willapa bar. The bar was not surveyed because of the presence of breaking waves, so surveyed depths on either side of the bar were averaged across the bar to give minimum depths of 6 m mtl. Persons knowledgeable about the entrance provided estimates of 1- to 2-m mtl depths in the breaking regions. After modifying the depths across the bar, reasonable agreement was found between STWAVE calculations and the field measurements. STWAVE was initialized with data from the Grays Harbor wave gauge, and the tide was represented with water elevations measured at Station 1.

Comparisons of the calculated and measured wave heights and directions are plotted in Figures 5-5 through 5-10. Statistics are presented in Table 5-6. The calculations tend to overestimate the wave height at low tide at Station 2, although the variation in wave height with the tide is reproduced. The wave directions are modeled well for Station 1, but the calculated directions at Stations 2 and 3 are less variable than measured. The wave directions at Station 2 are more out of the northwest at low tide, when energy from the west is dissipated over the bar and energy reaches Station 2 through the North Channel entrance. At Station 3, wave directions are more from the west at low tide because the energy reaching Station 3 is conveyed through the Middle Channel. The larger errors in wave direction at Station 2 for 12-14 November coincide with strong winds from the south and southeast (as great as 17 m/sec), which generate short-period waves from the south bay. These locally generated waves are not represented in the model. The large uncertainty in the bathymetry over the shoals that front Stations 2 and 3 (because of lack of direct measurements in the breaking regions and sand movement during the interval between the bathymetry surveys and wave measurements) contributes to the model error. Overall, the verification is considered to show reasonable agreement between the calculations and measurements for this energetic and complex environment.

Simulation of Design Alternatives

To evaluate Willapa channel alternatives, two types of wave model simulations were performed. Fifteen representative wave spectra were transformed for ebb, slack, and flood currents to evaluate navigation conditions in the channels. Then, a time-history of waves for a winter month was transformed to provide input for sediment transport estimates.

Table 5-6 Verification Statistics							
		Wave Height				Wave Direction	
Date	Sta	Mean Error percent	Mean Error m	rms Error percent	rms Error m	Mean Error deg	rms Error deg
9/1/98-9/4/98	1	6.6	0.10	11.0	0.22	-0.2	2.2
	2	35.1	0.20	45.1	0.23	8.1	14.7
	3	17.1	0.06	31.8	0.09	3.4	12.5
10/15/98-10/20/98	1	-2.4	-0.05	11.5	0.22	-6.7	7.7
	2	25.8	0.13	43.0	0.21	3.8	15.2
	3	17.4	0.07	32.2	0.12	13.3	19.8
11/11/98-11/16/98	1	-3.9	-0.13	11.0	0.37	-7.7	11.3
	2	22.5	0.18	35.3	0.27	15.4	74.3
	3	-22.1	-0.08	26.7	0.10	6.4	8.5

Waves to evaluate navigation

To evaluate the channel alternatives from the perspective of navigability, 15 representative waves were selected. These waves are the most commonly occurring in the wave climate with heights less than 3 m (occurrence greater than 1.5 percent), and account for 70 percent of all wave conditions (85 percent of all waves under 3 m). Only waves less than 3-m cutoff were considered because it is about the limiting wave height to navigate the entrance (see Chapter 2). The 15 waves include incident directions of 225 to 315 deg and periods of 8 to 16 sec. The waves are listed in Table 5-7 with their probability of occurrence.

The navigation evaluation considered five alternatives: Alternative 1 (existing condition), Alternative 3A (existing condition with dredging on the bar), Alternative 3B (North S-Channel), Alternative 4A (Middle Channel), and Alternative 3H (North Channel displaced to the south). Because the navigation evaluation focuses only on the bar channel, these five alternatives cover the range of all North and Middle Channel alternatives. Alterations of the interior channels (except Alternative 4D) resulted in only minor changes to currents in the bar channel (Chapter 6); thus, results for Alternative 4A are representative of 4B, 4C, and 4E. Simulation of the five alternatives included the following steps:

a. The bathymetry grid was modified for each channel alternative as discussed in Chapter 6.

b. Fifteen representative spectra were generated with heights, periods, and directions selected from the wave climate and spectral parameters given in Table 5-4.

c. Typical peak ebb and flood current fields were saved from ADCIRC simulations of each alternative.

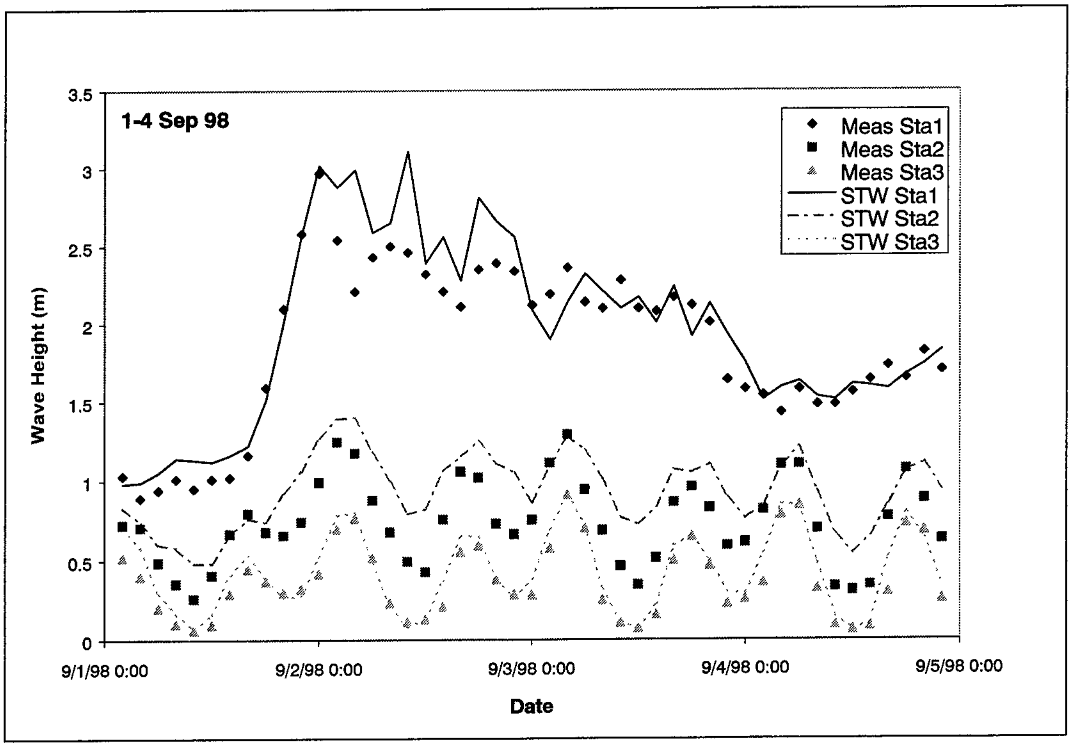

Figure 5-5. Wave height verification for 1-4 September 1998

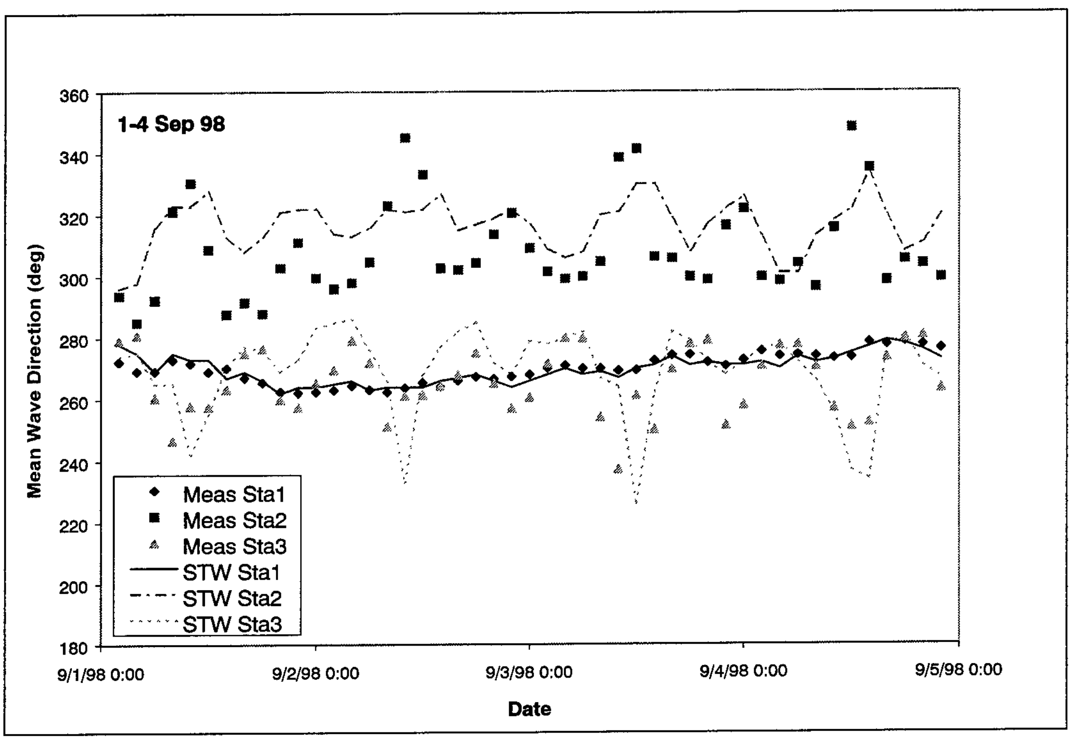

Figure 5-6. Wave direction verification for 1-4 September 1998

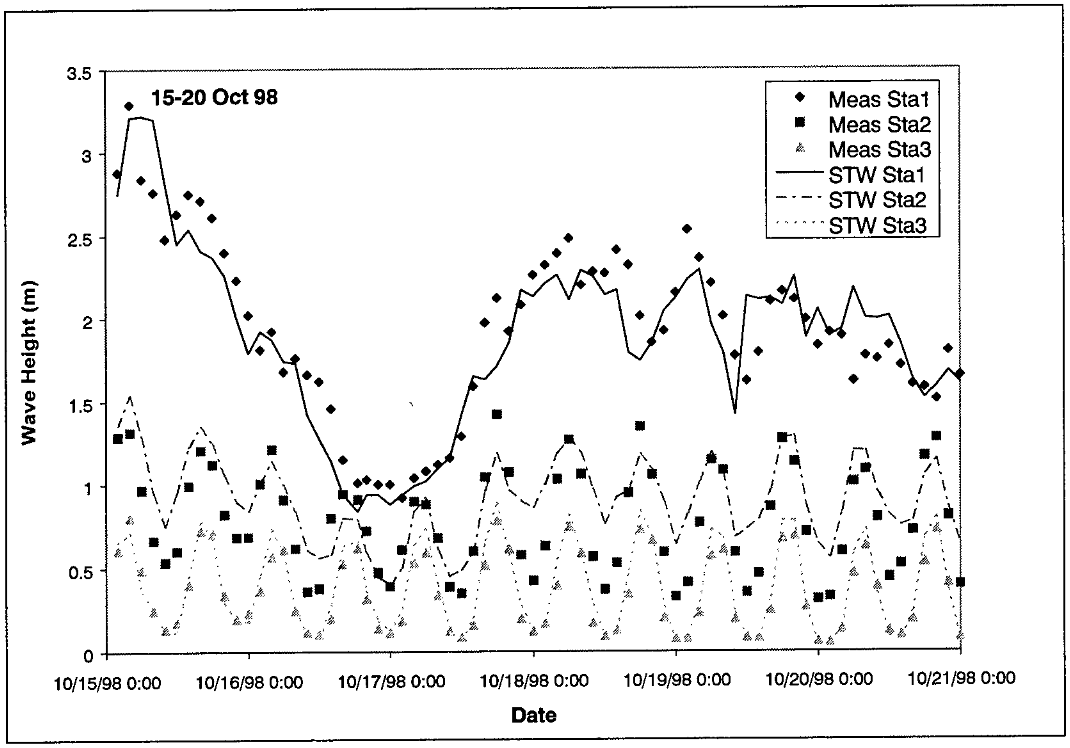

Figure 5-7. Wave height verification for 15-20 October 1998

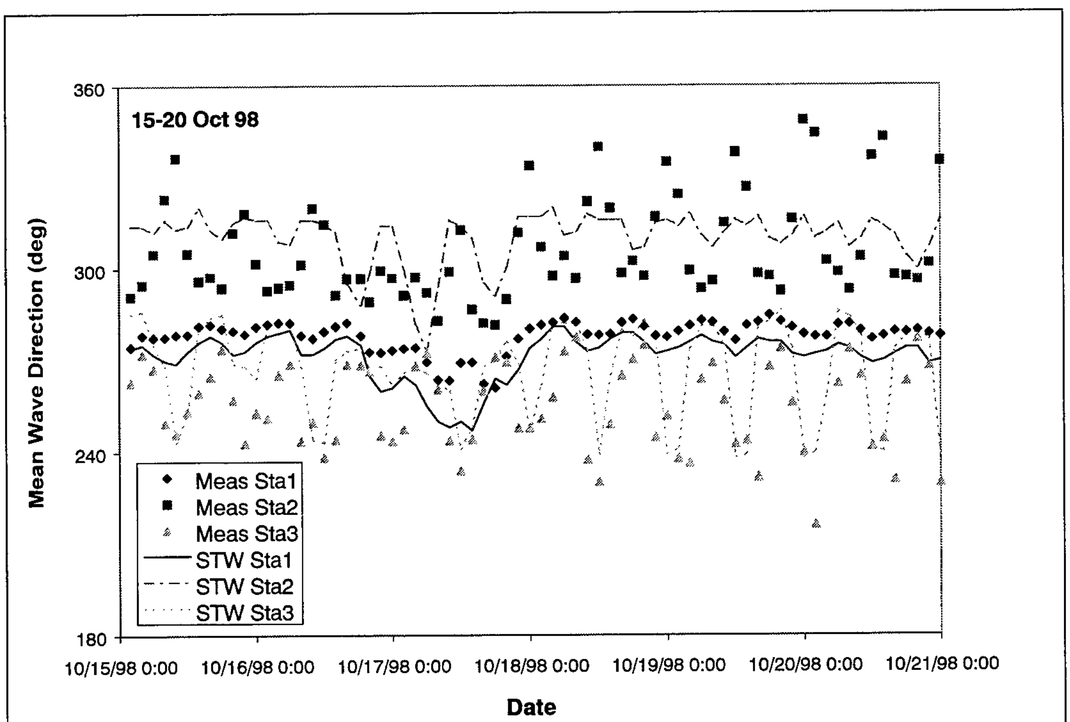

Figure 5-8. Wave direction verification for 15-20 October 1998

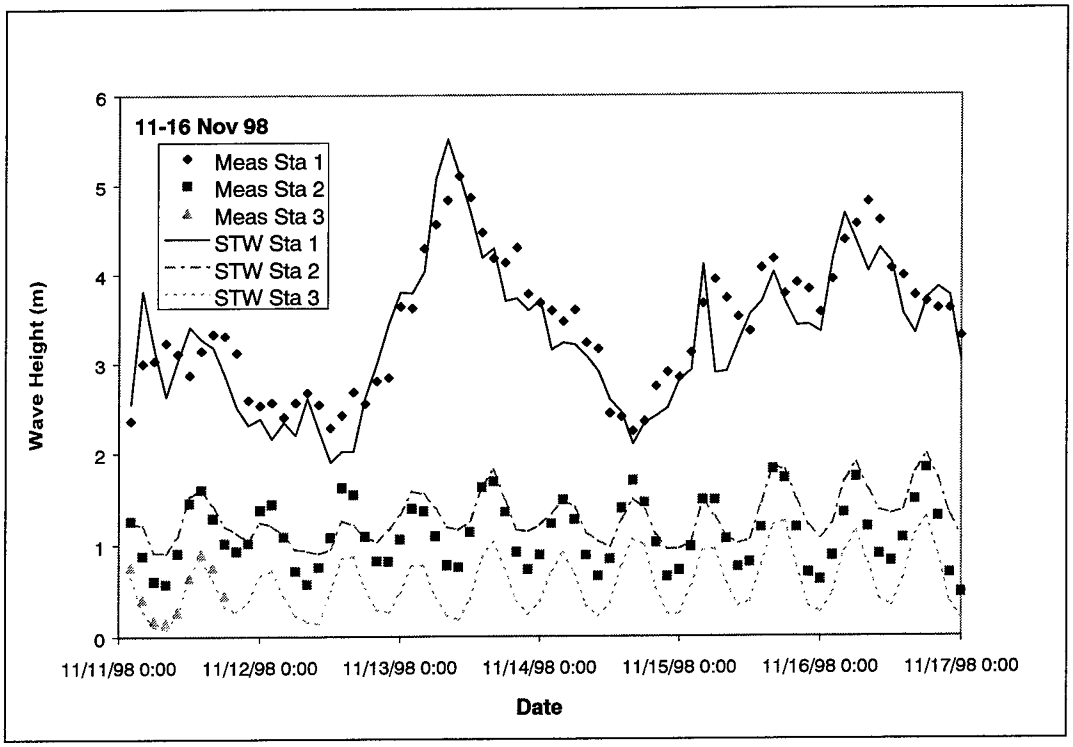

Figure 5-9. Wave height verification for 11-16 November 1998

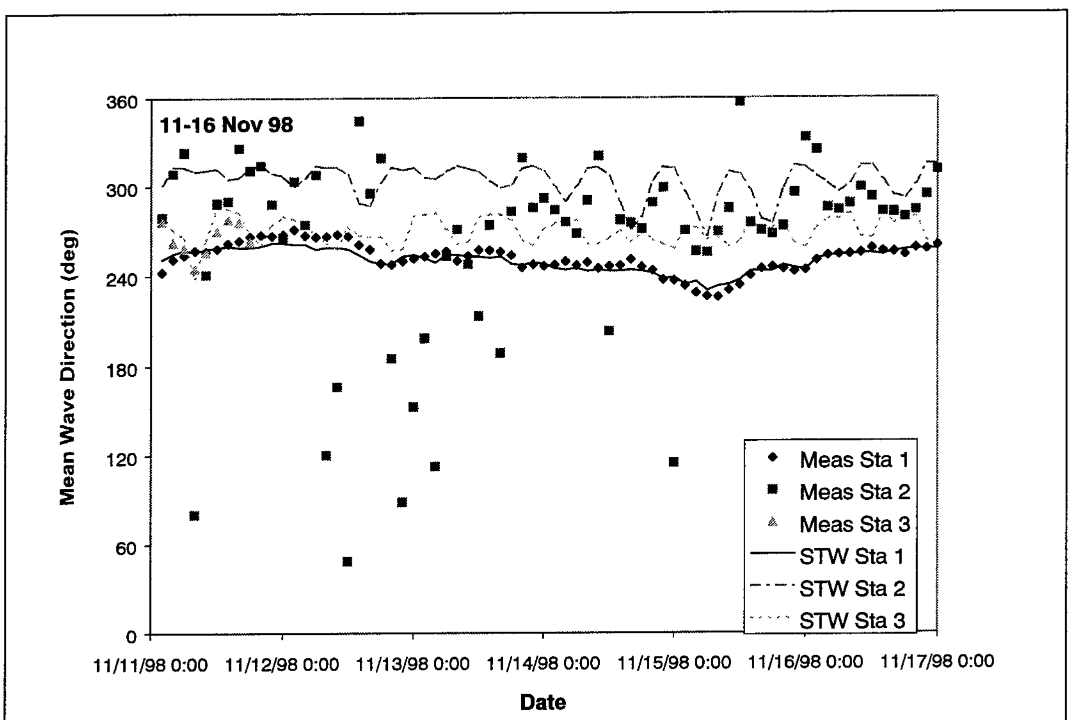

Figure 5-10. Wave direction verification for 11-16 November 1998

d. STWAVE was run to transform the 15 representative waves across each of the bathymetry grids for the five alternatives. Runs were made at mtl with no current, mlw (-1 m) with a typical peak ebb current, and mhw (1 m) with a typical peak flood current. The total number of model runs was 225 (15 waves times three tide/current combinations times five alternatives).

The purpose of the STWAVE navigation-evaluation runs is to compare wave heights and directions in the channels for the five alternatives to identify a preferred alternative. To compare the results, three regions were selected in the entrance channel as the outer channel, midchannel, and inner channel. The outer channel location is the seaward end of each channel, where the channel "daylights" (channel depth equals the longshore depth contour). The midchannel location is the most restricted channel cross section (narrowest width through the shallowest bar section). The inner channel location is bayward of the bar. The locations are separated by distances of 1-2 km. Examples of the wave heights and wave directions (relative to the channel orientation) for the most frequently occurring wave condition (Wave 4 in Table 5-7) are given in Figures 5-11 and 5-12, respectively. Wave heights and directions for ebb, slack, and flood are averaged (heights are generally higher for ebb and lower for flood). In the outer channel and midchannel, the wave heights are similar for the five alternatives. In the inner channel, Alternatives 3B and 3H have significantly higher wave height. The wave directions relative to the local channel orientation are within about ±20 deg in the outer channel and midchannel, but increase to over 60 deg for Alternative 3B and 30 deg for Alternative 4A in the inner channel. The 60-deg obliqueness relative to the channel orientation exceeds the 45-deg window for safe navigation discussed in Chapter 2.

Table 5-7
Representative Waves for Navigation Evaluation Simulations

Wave	H_{mo} m	T_p sec	α_m deg	Probability Percent
1	1.5	8	225.0	1.5
2	1.5	8	247.5	3.1
3	1.5	8	270.0	10.0
4	1.5	8	292.5	21.6
5	1.5	8	315.0	2.7
6	1.5	12	270.0	5.0
7	1.5	12	292.5	3.3
8	1.5	16	270.0	2.6
9	2.5	8	225.0	1.6
10	2.5	8	270.0	2.6
11	2.5	8	292.5	3.0
12	2.5	12	247.5	1.9
13	2.5	12	270.0	4.8
14	2.5	12	292.5	3.2
15	2.5	16	270.0	2.9

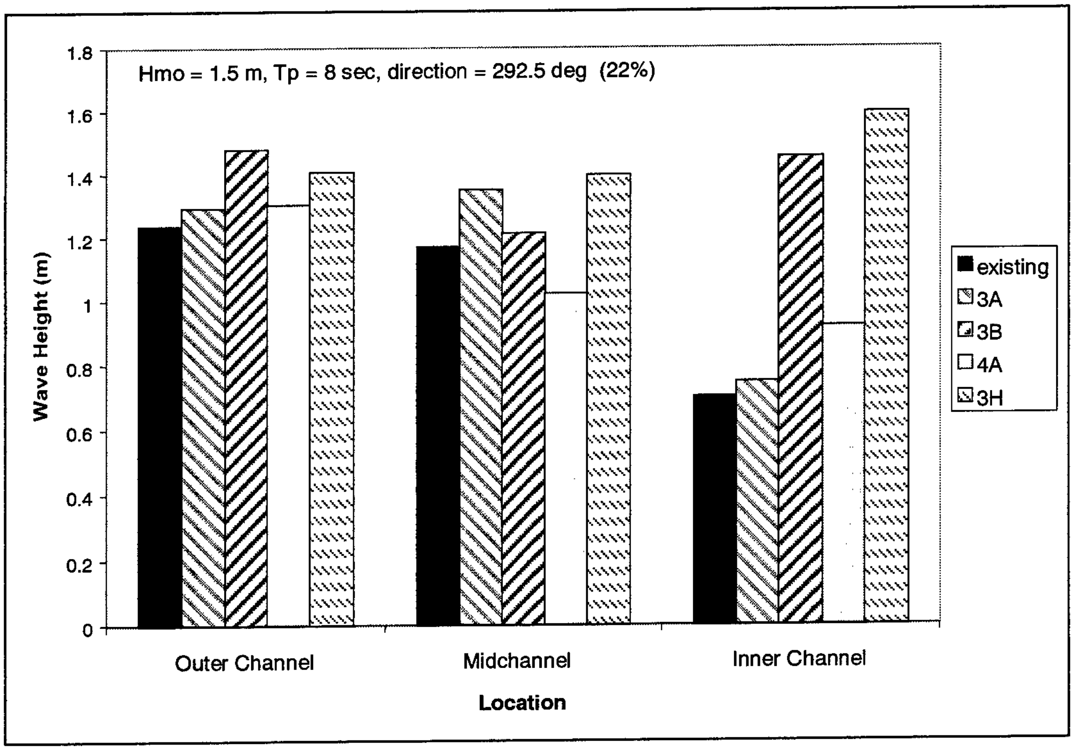

Figure 5-11. Wave height comparison for most frequent wave

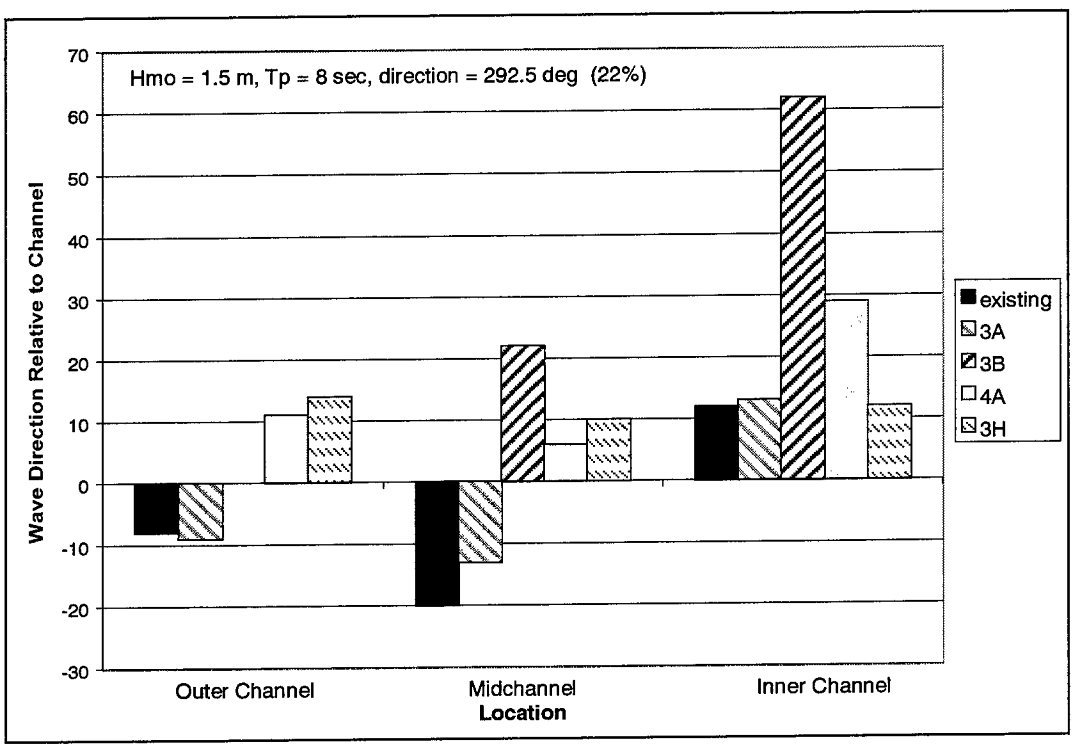

Figure 5-12. Wave direction comparison for most frequent wave

The obliqueness of waves in the inner channel for Alternative 3B is especially problematic because of the higher waves, thus greater wave steepness and potential for wave breaking. Results of averages from the 15 representative waves (weighted by their percent occurrence) are given in Table 5-8. Navigability for any alternative is dictated by the worst conditions that are encountered along the channel. Although Alternative 1 (existing conditions) provides the smallest wave heights, the channel depth is less than the 26-ft design depth (Chapter 2). Alternatives 3A and 4A are similar from the standpoint of waves for navigability. Alternative 3H has significantly higher wave heights in the inner channel than Alternatives 3A and 4A (wave height is approximately double), but has similar characteristics in the outerchannel and midchannel positions. Alternative 3B has poor wave height and direction characteristics in the inner channel.

Alternatives 1 and 3A. The statistics in Table 5-8 show that wave height and direction are fairly consistent between Alternatives 1 and 3A. This is reasonable because these alternatives have the same channel alignment, with a slightly deeper and wider channel in Alternative 3A. The wave height increases 10 to 15 percent for Alternative 3A relative to Alternative 1, because the deepening and widening of the entrance channel increase wave penetration. Wave angles relative to the channel exceed 45 deg in the outer channel or midchannel for about 10 percent of wave conditions less than 3 m (based on the 15 representative waves). Typically, incident wave directions of 247.5 deg and less result in angles greater than 45 deg relative to the outer channel and midchannel.

Alternative 3B. Alternative 3B (S-channel) has the poor characteristics in the inner channel of high wave heights (122 percent of Alternative 1) and wave angles oblique to the channel (51 deg, on average). Wave energy is focused on the inner channel by the entrance bar. Also in the inner channel, the channel orientation is toward the northeast/southwest (230 deg), so dominant incident waves from the west and west-northwest are oblique to the channel. A channel configuration similar to Alternative 3B was the marked channel until recently (October 1998). The difficulties in navigating this channel are the primary motivation of this study. Wave angles relative to the channel exceed 45 deg for about 60 percent of wave conditions less than 3 m. Typically, incident wave directions of 225 deg and less for the outer channel or 292.5 deg and greater for the inner channel result in angles greater than 45 deg relative to the channel.

Table 5-8
Comparison of Wave Height and Direction for Channel Alternatives

	Average H_{mo} M			Increase in H_{mo} Relative to Alternative 1 percent			Average • $_m$ Relative to Channel Orientation deg		
Alternative	Outer	Mid	Inner	Outer	Mid	Inner	Outer	Mid	Inner
1 existing	1.5	1.5	0.8	0	0	0	19	30	9
3A	1.6	1.7	0.9	9	15	9	20	23	10
3B	1.8	1.5	1.9	24	4	122	15	15	51
4A	1.7	1.4	1.1	17	-8	44	12	8	26
3H	1.7	1.8	1.9	14	10	158	14	10	11

Alternative 4A. Alternative 4A (Middle Channel) generally has higher wave heights in the outer and inner channels and lower heights in the midchannel compared with Alternatives 1 and 3A. The wave angles are generally less oblique in the midchannel, but more oblique in the inner channel. The reason for increased wave height and more oblique wave directions in the inner channel for Alternative 4A is that wave energy enters the region of the North Channel and is refracted and diffracted into the inner reach of the Middle Channel. For Alternatives 1 and 3A, wave energy is sheltered in the inner channel by Cape Shoalwater. Wave directions do not exceed 45 deg for the representative wave conditions.

Alternative 3H. Alternative 3H generally has similar wave heights and directions in the outer channel and midchannel to those of Alternatives 1 and 3A. But, the wave height is approximately double that of Alternatives 1 and 3A in the inner channel. The wave angles are generally slightly less oblique in the outer channel and midchannel. The reason for increased wave height in the inner channel for Alternative 3H is that wave energy is focused by the ebb shoal and interior shoals and there is less sheltering from Cape Shoalwater to the north (compared with Alternatives 1 and 3A). Wave directions do not exceed 45 deg for the representative wave conditions.

Navigability is a function not only of wave height and direction, but also of wave steepness (ratio of wave height to wavelength) and wave breaking. Steeper waves increase potential for a vessel to be overturned or swamped. Waves steepen in shallow water because wavelength decreases and height increases through refraction and shoaling. Similarly, waves steepen on ebb currents. For each of the alternatives, ebb and flood current magnitudes and channel depths are similar, so wave steepnesses are also similar (except the inner channel of 3B and 3H, where wave height and thus steepness can double). Wave breaking is also a function of wave height, wave steepness, and water depth. Thus, the wave-breaking characteristics of Alternatives 1, 3A, and 4A are expected to be similar. Alternatives 3B and 3H would experience increased breaking in the inner channel, where wave heights are higher than with the other alternatives. STWAVE can estimate regions of increased steepness or wave breaking, but not the timing or location of individual steep or breaking waves that might cause difficulty for navigation.

This study focused on waves in the bar channel, but results show that there are differences in wave height between the bay portions of the North and Middle Channels, also. At high tide (flood), waves are smaller in the bay portion of the North Channel than in the Middle Channel because of sheltering from Cape Shoalwater. At low tide (ebb), waves are slightly smaller in the bay portion of the Middle Channel because most of the wave energy is dissipated on the bar. Because vessels generally navigate through the outer bar on flood, near high tide, the North Channel alternatives offer decreased wave action in the interior bay channel, compared with that of the Middle Channel.

Detailed plots of the wave height and direction for the outer channel, midchannel, and inner channel for each alternative are given in Appendix D. Ebb, slack, and flood simulations for each wave are displayed separately.

Waves to evaluate sediment transport

To support sediment transport modeling, nearshore wave fields for one typical winter month were required. The month of January 1998 was selected because the Grays Harbor buoy record was continuous, and the mean wave height was near the mean winter wave height at the buoy (3.2 m for January 1998 versus 3.0 m for all winter months). A time-history of Grays Harbor measurements for January 1998 is given in Figure 5-13 (other winter months are plotted in Appendix C). STWAVE simulations were made with input every 3 hr from the Grays Harbor buoy, for a total of 248 model runs. Tide elevations were taken from the NOS Toke Point gauge. Tidal currents were neglected in the wave simulations because sensitivity tests showed that wave height and direction variation produced by the wave-current interaction were localized and would have minor influence on sediment transport calculations. The wave- and current-driven sediment transport calculations are discussed in Chapter 6.

Summary

The wave transformation model STWAVE was applied to calculate wave heights and directions at the entrance to Willapa Bay. The model resolution was 100 m and covered a domain of 30 by 51 km. The model was driven with input wave information from the Grays Harbor wave buoy. Field measurements taken at three stations in the mouth of Willapa Bay were used to evaluate the model for application to the complex environment of Willapa entrance. Verification results showed reasonable agreement between calculations and measurements at three locations where measurements were available. The model was applied to compare relative differences in wave height and direction for different channel alternatives.

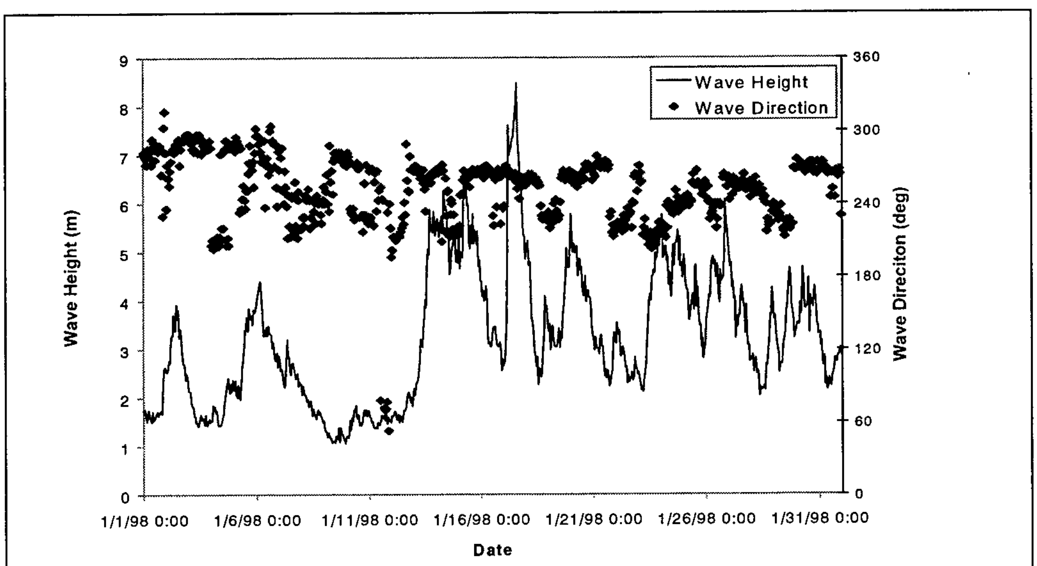

Figure 5-13. Grays Harbor measurements for January 1998

Following verification, five channel alternatives for Willapa Bay, Alternatives 1, 3A, 3B, 4A, and 3H, were evaluated. For each alternative, the model was run for 15 representative waves at ebb (mlw), slack (mtl), and flood (mhw). From the perspective of navigation, the following conclusions are made based on the short-wave modeling:

a. Alternatives 1 and 3A give equivalent results within the accuracy of the model. Wave heights for Alternative 3A are slightly higher (9-15 percent) because more wave energy can penetrate the deeper and wider entrance channel. Wave directions (relative to the channel axis) in the outer channel and midchannel exceed 45 deg for waves incident from the southwest and west-southwest. These North Channel alternatives have lower wave heights in the bay channel during high-tide conditions because wave energy is sheltered by Cape Shoalwater.

b. Alternative 3B gives consistently higher wave heights and more oblique wave directions (relative to the channel orientation) than Alternatives 1 and 3A. The higher waves also imply larger wave steepnesses and more frequent wave breaking. In the inner channel, wave directions would often exceed 45 deg, and typical wave heights would be near 2 m (compared with less than 1 m for Alternatives 1 and 3A), making Alternative 3B less preferable for navigation.

c. Alternative 4A gives results similar to those of Alternatives 1 and 3A, but the wave height is slightly higher in the outer and inner parts of the channel (17 and 44 percent, respectively, compared with Alternative 1), but lower in the midchannel (8 percent compared with Alternative 1). Wave angles tend to be less oblique than Alternative 1 in the midchannel, but more oblique in the inner channel. Waves at the inner channel for Alternative 4A are higher and more oblique because the Middle Channel is not naturally sheltered by Cape Shoalwater, and wave energy entering through the North Channel is refracted and diffracted around the Willapa bar to the inner channel region. Wave directions for the 15 representative waves do not exceed 45 deg in the Middle Channel.

d. Alternative 3H gives consistently higher wave heights in the inner channel compared with Alternatives 1 and 3A. The higher waves also imply larger wave steepnesses and more frequent wave breaking. The wave directions for Alternative 3H are generally less oblique to the outer channel and midchannel, than with Alternative 1 and 3A. Although the wave characteristics of Alternative 3H do not violate the navigation criteria, the larger interior wave heights would result in more difficult navigation over a longer distance in the entrance channel.

The STWAVE simulations do not show a clear preference for Alternative 1, 3A, or 4A based on navigation criteria of wave height and wave direction relative to the channel, within the accuracy of the model. Alternatives 3B and 3H are relatively less preferable because of the high waves (and oblique wave directions for Alternative 3B) in the inner channel.

Wave fields were also calculated for a typical winter month. STWAVE was run every 3 hr with input from the Grays Harbor buoy for January 1998. These STWAVE results are applied in Chapter 6 for sediment transport calculations.

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6 Circulation and Transport Modeling[1]

Numerical simulations of tidal circulation, sediment transport, and salinity at Willapa Bay were conducted to evaluate and compare engineering Alternatives for a safe and reliable entrance channel to Willapa Bay. This approach provides an objective means for comparing the performance of Alternatives. Details of the circulation, sediment transport, and salinity modeling conducted in this study are presented in this chapter.

Introduction

This chapter primarily describes implementation of the ADvanced CIRCulation (ADCIRC) and ADvanced TRANSport (ADTRANS) models. ADCIRC is a long-wave hydrodynamic model applied for simulating water-surface elevation and circulation over the entire bay as a function of tidal forcing, freshwater inflow, wave stress forcing, and wind forcing. Sediment transport for Willapa Bay was calculated within ADCIRC with a formulation for the total sediment load. ADTRANS is a model that calculates the concentration of specified parameters, in this case salinity, by application of the convection-diffusion equation.

Chapter details include a general description of the bay, circulation modeling approach, transport modeling approach, development of the computational grid, and results of the modeling efforts, as follows:

a. Physical properties of Willapa Bay are documented to provide a background for understanding the processes that force and define the system. The primary forcing for the bay is the tide, but wind and freshwater influx also contribute to motion in the bay.

b. Previous modeling studies of Willapa Bay are described.

c. Circulation modeling is presented. An overview of the ADCIRC model, information on boundary condition specifications, and bottom- and wind-stress formulations are presented.

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[1] Written by Dr. Adele Militello, Dr. Norman W. Scheffner, and Mr. Steven M. Bratos, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS; Mr. Mitchell E. Brown, Mevatec Corporation, Vicksburg, MS; and Mr. Scott Fenical, Pacific International Engineering PLLC, Seattle, WA.

d. Wave-driven current calculations are described and verified for a planar beach.

e. Sediment transport calculations are described and examined for reasonability. Formulations for the transport rate and related parameters set in the algorithm are given.

f. The salinity transport method is presented, as well as an overview of the ADTRANS model and information on boundary condition specifications.

g. Local and global features of the computational grid are described. Grid details presented are the computational domain, bathymetry, and resolution. The Alternatives and the State Highway Route (SR) 105 dike and groin structure are presented in detail.

h. Model calculations are compared with measurements of water level and current velocity.

i. Alternatives are evaluated based on current speed and sediment deposition within the channels.

Physical Setting of Willapa Bay

Physical properties of Willapa Bay are presented here to provide background information. Circulation and salinity transport in Willapa Bay are driven by the tide, freshwater inflows, and wind. Sediment transport is a function of these processes as modified by short (wind-generated) waves, especially at the entrance. Continental shelf processes outside the bay, such as the Columbia River freshwater plume and upwelling (Sternberg 1986; Landry et al. 1989; Hickey 1989), play a role in flow and transport inside the bay. Additionally, episodic events such as winter storms and processes related to El Niño can modify the circulation, thereby changing sediment and salinity transport.

Tide

Diurnal and semidiurnal constituents are the dominant contributors to tidal motion in Willapa Bay. The primary constituents forcing the bay are M_2, K_1, S_2, O_1, and P_1, in order of largest to smallest amplitude. The M_2 tidal constituent is dominant, being 2.3 times greater than the K_1 constituent at Toke Point. Table 6-1 gives tidal constituent amplitudes for water level at Toke Point, South Bend, and Nahcotta as calculated by the National Ocean Service (NOS), National Oceanic and Atmospheric Administration (NOAA). Tidal amplitudes increase from the entrance into the interior of the bay. The M_2 amplitude is 14 percent and 20 percent greater at South Bend and Nahcotta, respectively, than at Toke Point, according to NOS-calculated constituents based on measurements of water level.

Table 6-1
Tidal Constituent Amplitudes, m, at Toke Point, South Bend, and Nahcotta (NOS 1991 http://www.opsd.nos.noaa.gov)

Constituent	Toke Point	South Bend	Nahcotta
M_2	0.98	1.12	1.18
S_2	0.26	0.30	0.29
N_2	0.20	0.23	0.23
K_1	0.43	0.43	0.44
M_4	0.02	0.06	0.07
O_1	0.26	0.26	0.26
M_6	0.01	0.03	0.03
NU_2	0.04	0.05	0.04
MU_2	0.01	--	--
$2N_2$	0.02	0.03	0.03
OO_1	0.01	0.01	0.01
LDA_2	0.01	0.01	0.01
S_1	0.01	--	--
M_1	0.01	0.02	0.02
J_1	0.02	0.02	0.02
MM	0.02	--	--
MSF	0.03	--	--
MF	0.04	--	--
RHO_1	0.01	0.01	0.01
Q_1	0.04	0.05	0.05
T_2	0.02	0.02	0.02
R_2	0.00	0.00	0.00
$2Q_1$	0.01	0.01	0.023
P_1	0.13	0.14	0.14
L_2	0.02	0.03	0.03
K_2	0.06	0.08	0.08
S_4	--	0.01	0.01
M_8	--	0.00	0.01

Tidal range is defined as the difference in height between consecutive higher high and lower low water levels. At Toke Point the tide range has a mean value of 2.7 m, but varies considerably during the year and can reach almost 4 m during a strong spring tide. The tide range increases from the entrance to the upper reaches of the bay because of the amplification of the tidal wave as it propagates into the bay. Tidal datums computed at gauges located in Willapa Bay are given in Table 6-2. Values are referenced to mean lower low water (mllw).

Table 6-2
Tidal Datums, m, for Willapa Tide Gauges (NOS 1998)

Datum	Toke Point	South Bend	Nahcotta	Naselle River
Highest observed water level	4.4	3.4	3.5	3.8
Mhhw	2.7	3.0	3.1	3.3
Mhw	2.5	2.8	2.8	3.0
Mtl	1.4	1.6	1.6	1.7
Mlw	0.4	0.4	0.4	0.5
Mllw	0.0	0.0	0.0	0.0
Lowest observed water level	-1.2	-1.2	-1.0	-1.3

Note: mhhw = mean higher high water, mhw = mean high water, mtl = mean tide level, mlw = mean low water, mllw = mean lower low water.

Freshwater inflows

Stream tributaries to Willapa Bay include the Willapa, Naselle, North, Bear, Nemah, Palix, and Cedar Rivers, and Smith Creek. Discharge data for May through December 1998 were obtained from the U. S. Geological Survey (USGS) for the Naselle and Willapa Rivers and are plotted in Figures 6-1 and 6-2, respectively. Both rivers had low discharges from May through October. Discharges increased significantly in November and December, following seasonal trends in precipitation. Peak discharges were approximately 120 m^3/sec greater for the Willapa than for the Naselle.

Local wind

Meteorological data for the Washington coast near Willapa Bay were compiled by Sternberg (1986). Winds are typically directed toward the north to northeast during October through April, and toward the south to southeast during May through September. The strongest winds occur during November through February, with wind speeds on the order of 5 m/sec for fair weather. Because of the large tidal range of Willapa Bay, local fair-weather wind forcing does not contribute significantly to water-level change in the bay.

Salinity

Coastal ocean salinity near Willapa Bay varies during the year as a function of wind forcing and northward movement of the Columbia River plume. According to Landry et al. (1989), from May to August, nearshore salinity typically ranges from 30 to 32 parts per thousand (ppt), with a maximum occurring in June. During the remainder of the year, nearshore salinity is in the range of 29 to 30 ppt. Because of strong Columbia River discharges during the summer coupled with seasonal winds typically directed toward the south, the

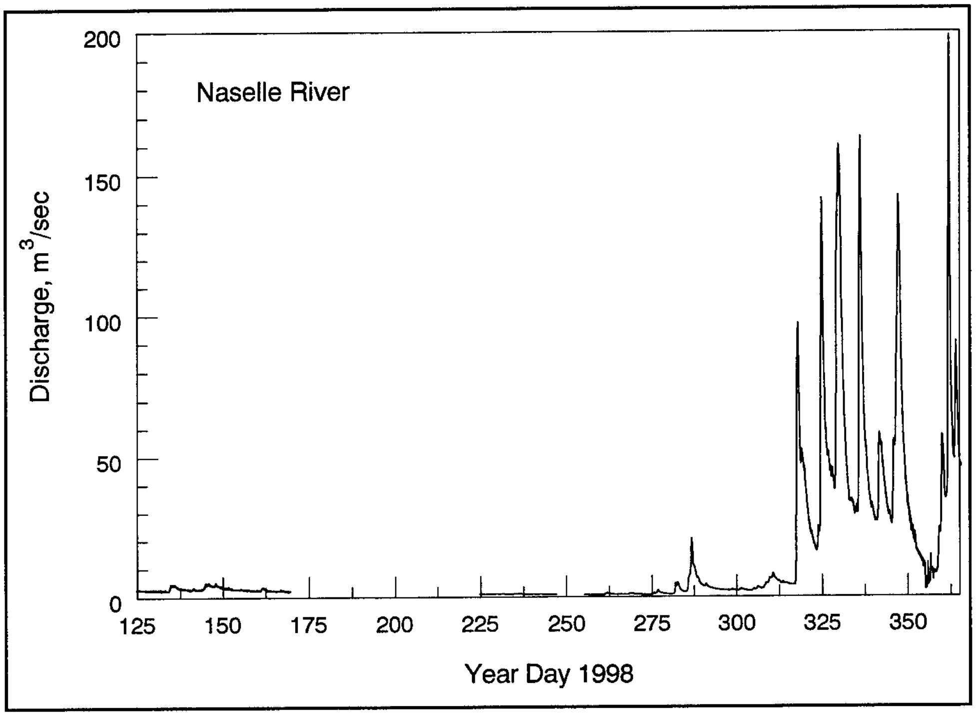

Figure 6-1. Naselle River discharge for May through December 1998

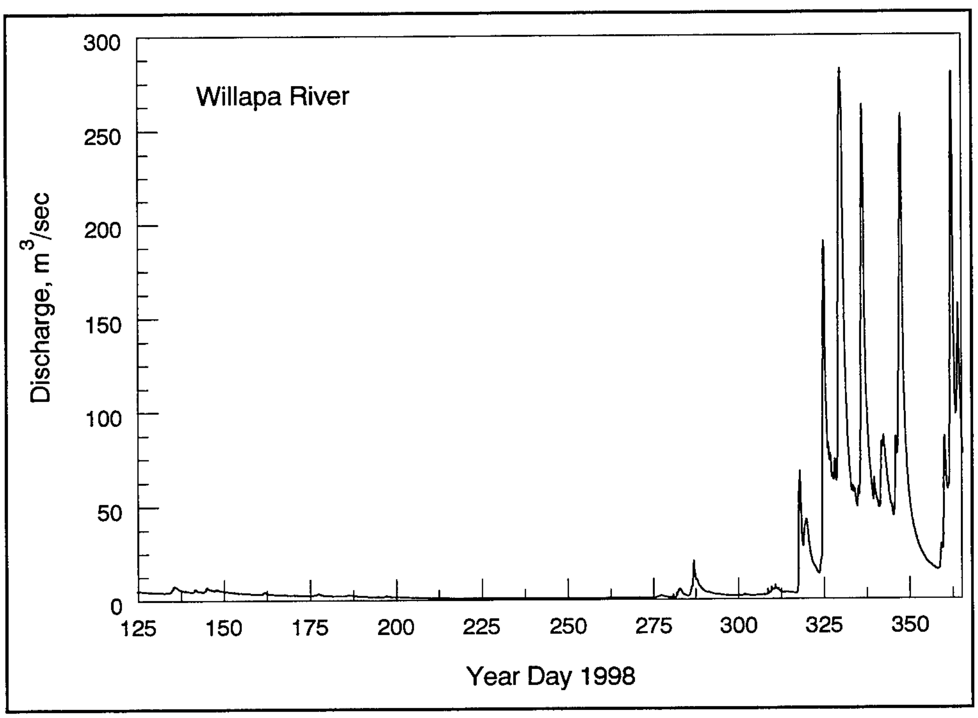

Figure 6-2. Willapa River discharge for May through December 1998

Columbia River plume manifests itself as a low-salinity surface feature that lies offshore and south of the river discharge at the Oregon–Washington border. During the remainder of the year, northward-directed winds force the plume north and against the shoreline (Landry et al. 1989). Low-salinity water from the Columbia River can enter Willapa Bay, decreasing bay salinity.

Inside the bay, variability in the distribution of salinity occurs because of freshwater discharge from rivers. For example, in 1965 salinity at Nahcotta varied from 15 to 30 ppt, whereas at the Naselle River, the variation was from 5 to 29.5 ppt (U.S. Army Engineer District, Seattle, 1971). Long-term (1961-1987) monitoring data collected by the Washington State Department of Fish and Wildlife in the southern part of the bay indicate that the average monthly surface-layer salinity varies from 15 ppt in February to 29 ppt during August and September.[2] Monthly mean salinity values in the Willapa River near Toke Point calculated from measurements taken from 1990 through 1997 show a seasonal trend, with lowest salinity in December, increasing through July, and peak values occurring during July through November (Newton et al. 1998). In general, the bay is vertically well mixed during low tributary flows (May to October) and alternates between vertically well mixed and partly mixed during strong tributary flows (November through April) (U.S. Army Engineer District, Seattle 1971).

The total freshwater contribution into the bay, even during high tributary flows, represents less than 5 percent of the tidal prism, estimated to be on the order of 3.74×10^8 m^3 for a mean tidal range (Johnson 1973). During times of low discharge, salinity is nearly uniform over the water column, but decreases from the entrance to the interior along the major channels. This behavior is observed in vertical profiles of salinity measured along transects in the Willapa River and Nahcotta Channel taken during May-June 1995.[3] Vertical stratification was observed near and within tributaries, even during weak discharge. Salinity profiles conducted for this study during November 1998 (Chapter 4) showed similar behavior. Strong vertical stratification occurred in the Willapa and Naselle Rivers, and weak or no vertical stratification occurred in the central bay. In general, stratification was stronger during low slack tide and weaker during high slack.

Because of the presence of a major oyster industry in Willapa Bay, there is concern about long-term salinity change in the bay. Both Pacific and native adult oysters thrive in salinity greater than about 20 ppt.[4] Therefore, salinity changes that may arise from proposed Alternatives are of interest.

Previous Modeling Studies

Pacific International Engineering[PLLC] (PIE) and PHAROS Corporation performed a two-dimensional (2-D) hydrodynamic modeling study of Willapa Bay for the Washington Department of Transportation (WDOT) to evaluate a proposed project to stabilize the SR-105 near North Cove of Willapa

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[2] Data provided by Mr. Bruce Kauffman, Willapa Field Station, Washington State Department of Fish and Wildlife, Ocean Park, WA, 19 November 1998.

[3] Plots furnished by Dr. Barbara Hickey, University of Washington, Seattle, WA, 1998.

[4] Personal Communication, Mr. Brett Dumbauld, Willapa Field Station, Washington State Department of Fish and Wildlife, Ocean Park, WA, 4 December 1998.

Bay (PIE and PHAROS Corporation 1997). The study applied the FESWMS-2DH model together with a finite-element grid of the bay. The grid had 3,732 nodes and 1,180 elements and primarily covered the interior of the bay. The model was calibrated with velocity measurements. Flow modifications from construction of a North Channel plug together with dredging of the Middle Channel were studied by comparing resultant calculations for preproject and postproject velocity values. The study concluded that the proposed project changed the depth-averaged maximum currents in the North and Middle channels, but had no significant flow modifications elsewhere in the bay.

Circulation Model

Water-surface elevation and current for Willapa Bay were calculated by the 2-D depth-integrated, hydrodynamic model ADCIRC (Luettich, Westerink, and Scheffner 1992). The ADCIRC model is a finite element hydrodynamic model developed under the U.S. Army Corps of Engineers Dredging Research Program. The finite element formulation has the advantage of great flexibility in resolution over the calculation domain. Coarse resolution can be specified in areas distant from the local region of interest, and fine resolution can be specified locally to meet project requirements. For instance, channels and structures can be defined for accurate calculation of flow through and around them.

Specifications for ADCIRC simulations of hydrodynamics in Willapa Bay include forcing with tidal constituents, forcing with wind, wetting and drying, calculation of nonlinear continuity and advection, quadratic bottom stress, and quadratic wind stress. Open-ocean boundaries were forced by tidal constituents obtained from the Le Provost et al. (1994) database. Wind data obtained from the National Center for Environmental Prediction (NCEP) were applied as meteorological forcing for simulations. ADCIRC has robust wetting and drying algorithms that were applied to simulate the inundation and exposure of expansive tidal flats in Willapa Bay. Bottom friction was specified for the Willapa Bay simulations as a quadratic bottom-stress formulation given by

$$\tau_* = \frac{C_f \left(U^2 + V^2\right)^{1/2}}{D} \tag{6-1}$$

where

C_f = friction coefficient specified as 0.0025 for the Willapa Bay simulations

U = current velocity along the x-axis

V = current velocity along the y-axis

D = total water depth

Within ADCIRC, flow toward the east is taken as positive along the x-axis and flow toward the north is taken as positive along the y-axis. Wind stress given in terms of direction components τ_x and τ_y is

$$\tau_x = C_{10}\rho_a |W| W_x \tag{6-2}$$

$$\tau_y = C_{10}\rho_a|W|W_y \tag{6-3}$$

where

C_{10} = wind-drag coefficient for wind at 10 m above the water surface

ρ_a = density of air

W = wind velocity

W_x = x component of the wind velocity

W_y = y component of the wind velocity

Wind velocities are at 10 m above the water surface. The wind-drag coefficient is calculated as (Garrett 1977)

$$C_{10} = (0.75 + 0.067 \times 10^{-2}\ W) \times 10^{-3} \tag{6-4}$$

where wind speed is specified in centimeters per second.

Wave-driven currents

To meet the objectives of this study, the ADCIRC model was upgraded to include wave-driven currents by implementation of wave stresses in the momentum equations. Spatial gradients of radiation stresses were calculated from wave parameters computed by STWAVE (see Chapter 5). These wave stresses were added into the momentum equations in a form analogous to the wind stress.

Wave stresses R_x and R_y, corresponding to the x- and y-directions, respectively, are given by

$$R_x = \frac{1}{\rho}\left[\frac{\partial S_{xx}}{\partial x} + \frac{\partial S_{xy}}{\partial y}\right] \tag{6-5}$$

and

$$R_y = \frac{1}{\rho}\left[\frac{\partial S_{xy}}{\partial x} + \frac{\partial S_{yy}}{\partial y}\right] \tag{6-6}$$

where ρ is the density of water, and S_{xx}, S_{xy}, and S_{yy} are radiation stresses. By applying linear wave theory, the radiation stresses are represented as

$$S_{xx} = E\left[n\left(\cos^2\theta + 1\right) - \frac{1}{2}\right] \tag{6-7}$$

$$S_{xy} = \frac{E}{2} n \sin 2\theta \tag{6-8}$$

$$S_{yy} = E\left[n\left(\sin^2\theta + 1\right) - \frac{1}{2}\right] \tag{6-9}$$

where

E = wave energy

n = ratio of group velocity to wave celerity

θ = wave angle

For more information on linear wave theory, see Dean and Dalrymple (1992).

Verification of Wave Stress Forcing

Wave-stress forcing was verified by comparing radiation-stress values calculated from STWAVE wave output to those calculated by the longshore current model Numerical Model of LONGshore current (NMLONG) (Larson and Kraus 1991) and with an analytical solution. A planar beach was specified to provide an idealized situation for model comparison. Wave-stress forcing within ADCIRC was validated by comparison of the calculated wave-driven current with measurements made on Leadbetter Beach, California (Thornton and Guza 1986).

Radiation-stress gradients computed from STWAVE-calculated wave parameters were evaluated by simulating wave propagation over a planar beach with a 1V on 30H slope. The input wave conditions were significant height of 2.0 m, period of 11.5 sec, and direction with respect to normal of –30 deg. Figure 6-3 shows plots of the wave-stress vector field, wave-stress magnitude, wave height, depth, and wave direction for the nearshore portion of the profile. The wave refracts to an angle of –10 deg and breaks at x = 2,400 m at depth of 3.8 m. After the wave breaks, the wave height decays as the wave propagates across shore. After breaking there is also a significant increase in wave stress from near zero to 0.06 m^3/sec^2 at x = 2,420 m followed by a more gradual decrease to zero as the wave propagates to the shore. The wave-stress field shows that the wave stresses in the surf zone are directed primarily toward shore, but contain a small alongshore component.

Radiation stresses computed from STWAVE-calculated wave parameters were compared with those calculated by NMLONG. Figure 6-4 shows the root-mean-square (rms) wave height H_{rms}, radiation-stress magnitude S, and the radiation stress components S_{xx} and S_{xy} for each model along the profile. Radiation-stress values shown in the figure have been normalized by the density of seawater. STWAVE and NMLONG apply different methods for calculating wave propagation and breaking; therefore, wave heights calculated along the profile by the two models are not quite equal. Because the radiation-stress components are proportional to the square of the wave height, radiation stresses calculated by each model differ by a corresponding amount. Peak rms wave height calculated by NMLONG is 1.5 m, a value that is comparable to 1.4 m

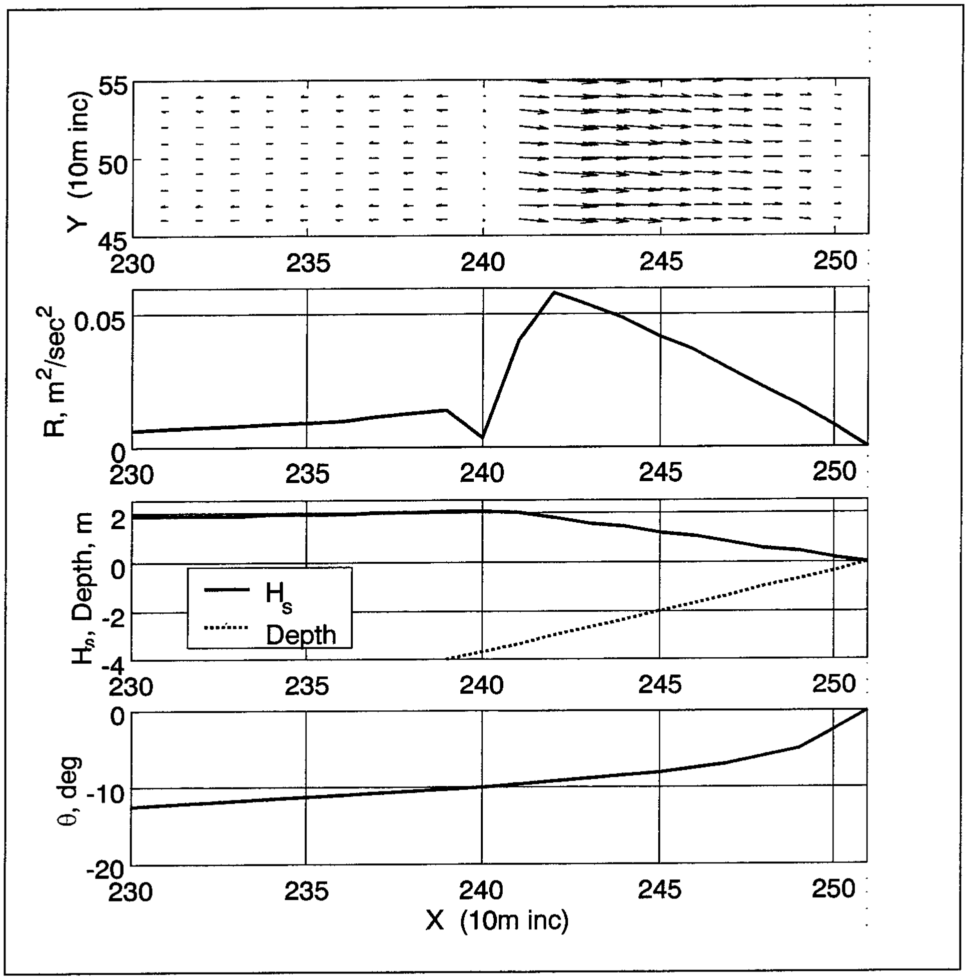

Figure 6-3. Wave parameters calculated by STWAVE with input parameters of H_s = 2.0 m, T = 11.5 sec, and θ = -30 deg for a planar beach (lV:30H). Panels from top to bottom are wave stress vector field, wave stress magnitude R, significant wave height and water depth, and wave angle

calculated by STWAVE. The peak NMLONG-calculated rms wave height occurs further offshore, at x = 2,350 m, than that calculated by STWAVE, which is located at x = 2,400 m. The NMLONG-calculated rms wave height has a broader distribution across the surf zone than that calculated by STWAVE. The rms wave height calculated by STWAVE is greater than that calculated by NMLONG in the area where the STWAVE-calculated rms wave height is decaying. Differences in radiation-stress magnitude and components between the two models correspond to those in rms wave height. Despite the difference in wave propagation and breaking methods, the models produce similar results for rms wave height and radiation stress.

Further evaluation of radiation stress calculated by STWAVE output was conducted by comparison to analytical solution of radiation stress at locations along the profile. Local wave conditions for the analytic solution were calculated

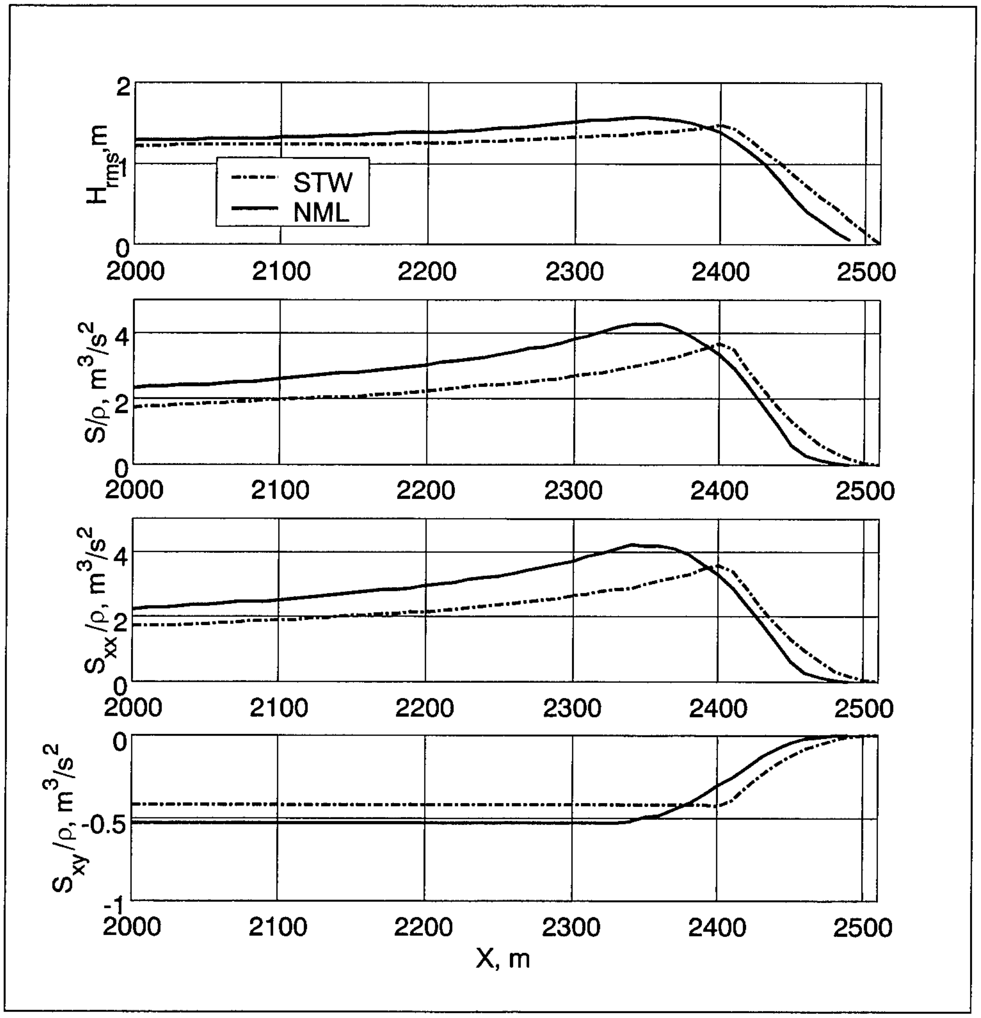

Figure 6-4. Comparison of STWAVE- and NMLONG-calculated wave parameters. Panels from top to bottom are rms wave height, radiation stress magnitude S/ρ, and the magnitude of radiation stress components, S_{xx}/ρ and S_{xy}/ρ

with linear wave theory. The objective was to produce local wave conditions similar to those calculated from STWAVE-calculated wave parameters. This objective was accomplished by adjusting the incident deepwater wave height in the linear calculations until the resulting local wave conditions were similar to those of the model for a particular depth. Table 6-3 shows that the radiation stress values calculated from STWAVE output (referred to herein as STWAVE radiation stress) in the cross-shore direction due to the cross-shore momentum S_{xx} are within 4 percent of the analytical values when the local wave conditions are similar.

Radiation-stress gradients for Willapa Bay were calculated from values of H_s, T, and θ as given by STWAVE. Radiation-stress components were calculated with Equations 6-7 through 6-9 substituting the rms wave height, $H_{rms} = 0.707\ H_s$, for the energy-based calculation. The wave-stress components were then calculated with Equations 6-1 and 6-2. Wave-induced currents were

Table 6-3
Radiation Stress Values Calculated Analytically from STWAVE-Calculated Wave Parameters, and by NMLONG[1]

Linear Theory				Radiation Stress S_{xx}/ρ, m^3/sec^2		
H_s M	θ deg	C_g/C	Local Depth m	Analytical	STWAVE	NMLONG
2.09	8.80	0.968	3.21	3.78	-	3.80
2.09	9.43	0.963	3.67	3.75	3.61	-
2.23	10.59	0.954	4.67	4.18	-	4.20

[1] Significant figures do not imply accuracy, but are given for comparison of calculation method.

simulated by including the STWAVE wave-stress components in the ADCIRC momentum equations. The wave-stress components were interpolated from the STWAVE finite-difference grid to the ADCIRC finite-element grid by means of a bilinear interpolation algorithm.

The wave stresses implemented in ADCIRC were evaluated by comparison to field measurements from the National Sediment Transport Study made at Leadbetter Beach, California (Thornton and Guza 1986). This evaluation was conducted by calculating wave stresses in STWAVE for Leadbetter Beach and applying those stresses within ADCIRC. Currents calculated by ADCIRC were then compared to measurements. Input spectra for STWAVE were derived from a significant wave height of 0.78 m, peak period of 14 sec, peak wave direction of 9 deg, spectral peakedness factor of 3.3, and directional spreading coefficient nn = 8. The spectral peakedness factor controls the width of the frequency spectrum where smaller values correspond to broader peaks. The energy in the frequency spectrum is spread in direction proportional to $\cos^{nn}(\theta - \theta_m)$, where θ is the direction of the spectral component and θ_m is the mean wave direction. Bathymetry in the STWAVE grid was represented as a plane sloping beach of 1V on 26H with a constant depth of 3.84 m offshore and a grid spacing of 5 m along- and across-shore. Figure 6-5 shows a comparison of the measured and STWAVE-calculated rms wave height.

An ADCIRC grid developed for Leadbetter Beach is shown in Figure 6-6. Node spacing varied between 10 and 2.5 m from the offshore boundary to the shoreline. The grid extended 185 m offshore and 200 m alongshore. Water level at the offshore boundary was held constant. The shore and lateral boundaries were closed and given a no-slip condition. This simulation was run for 12 hr, with a ramp of approximately 2.5 hr. The friction coefficient was set at 0.004, and the mixing coefficient was set at 0.3. The parameter τ_0, which controls the weighting of the generalized wave continuity equation between the pure wave equation ($\tau_0 = 0$) and the primitive continuity equation ($\tau_0 = 1$) (Luettich, Westerink, and Scheffner 1992), was set to 1.0 to give full weight to the primitive continuity equation for this very shallow-water calculation.

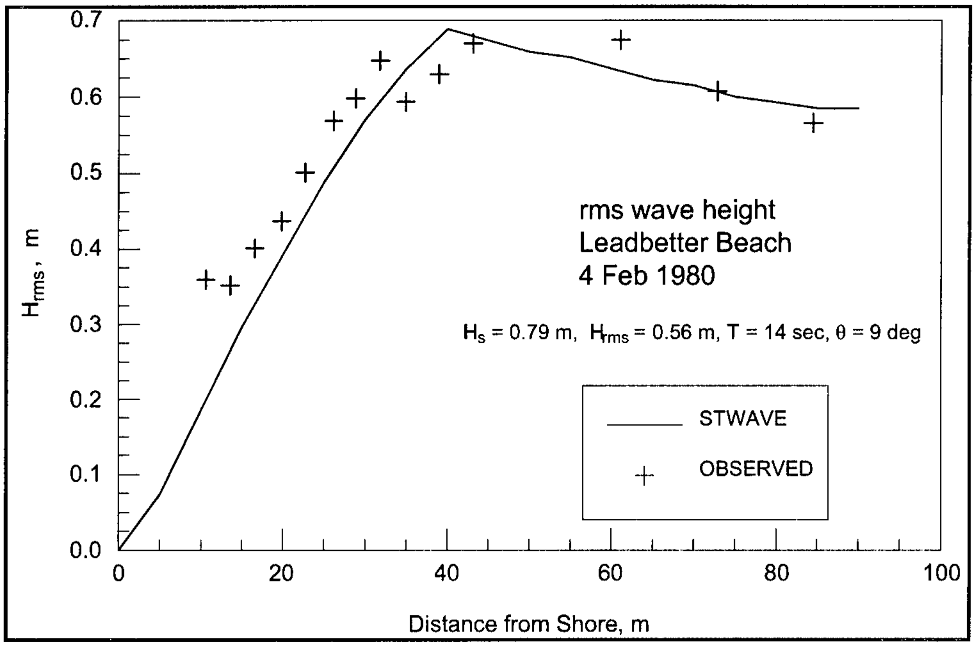

Figure 6-5. Comparison of measured and calculated rms wave height for Leadbetter Beach, California, 4 February 1980

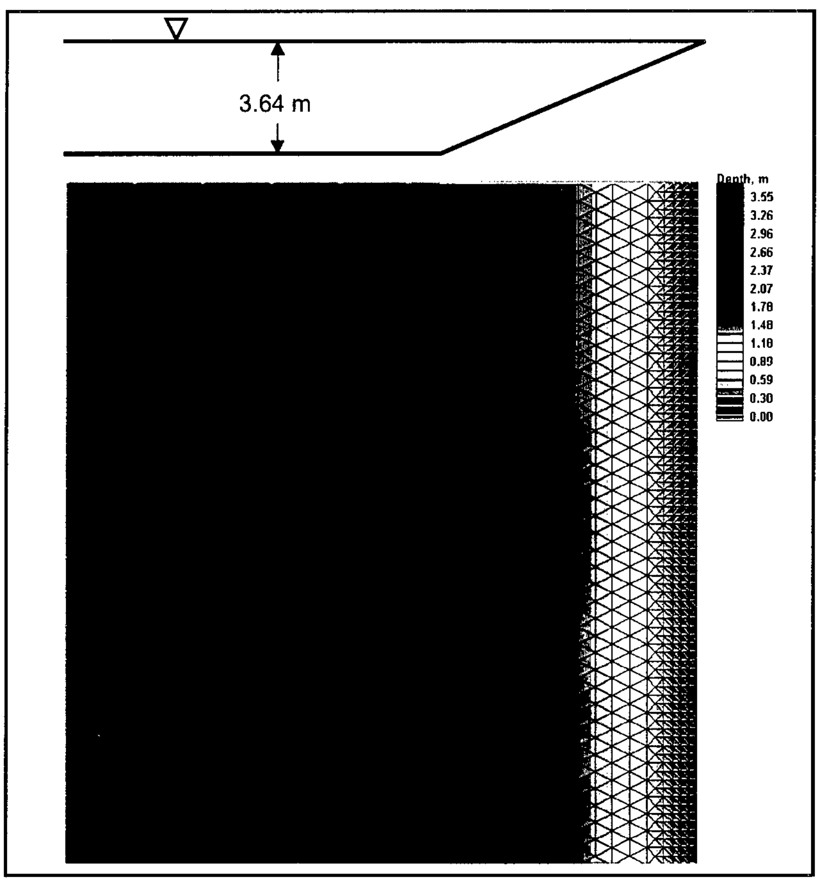

Figure 6-6. ADCIRC grid and bathymetry for Leadbetter Beach

A comparison between the calculated and measured longshore current v is shown in Figure 6-7. The model values are in good agreement with the measurements except for an overprediction in the area just outside the surf zone. The calculated peak longshore current is 5 percent above that of the measurement. From the breaking point at $x = 40$ m to where the longshore current peaks, the model overestimates the measurements by about 15 percent. Just offshore of the breaking point the model overestimates the longshore current by 70 percent compared to the measured value (in a region of small current magnitude).

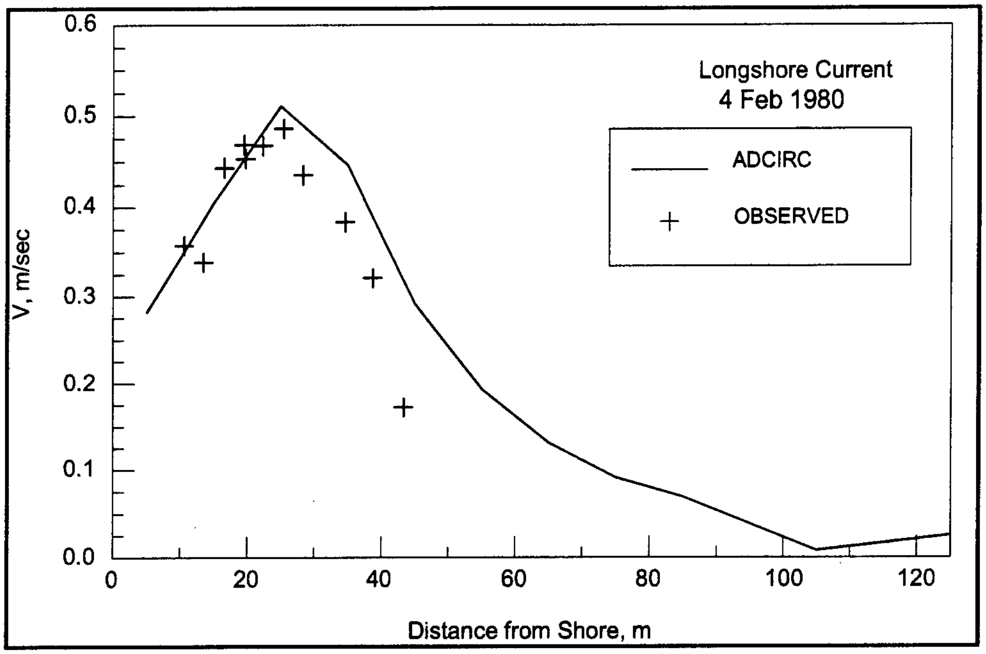

Figure 6-7. Calculated and measured longshore current at Leadbetter Beach, California, 4 February 1980

A second ADCIRC simulation was run for 60 hr with the same wave condition, but for a wave direction of 0 deg. Normal wave incidence is expected to produce an equilibrium condition with a time-invariant water level (setup-setdown), and no current. Figure 6-8 shows the water-surface elevation for four cross-shore locations. Equilibrium was reached in approximately 3 hr of simulation time and transient oscillations were present for another 15 to 20 hr. The water-surface elevation located 125 m from the shoreline in the constant depth area is zero. Outside the surf zone at a distance of 45 m, the setdown is approximately 0.01 m. Across the surf zone, the setup increases to approximately 0.075 m at the 5-m cross-shore location.

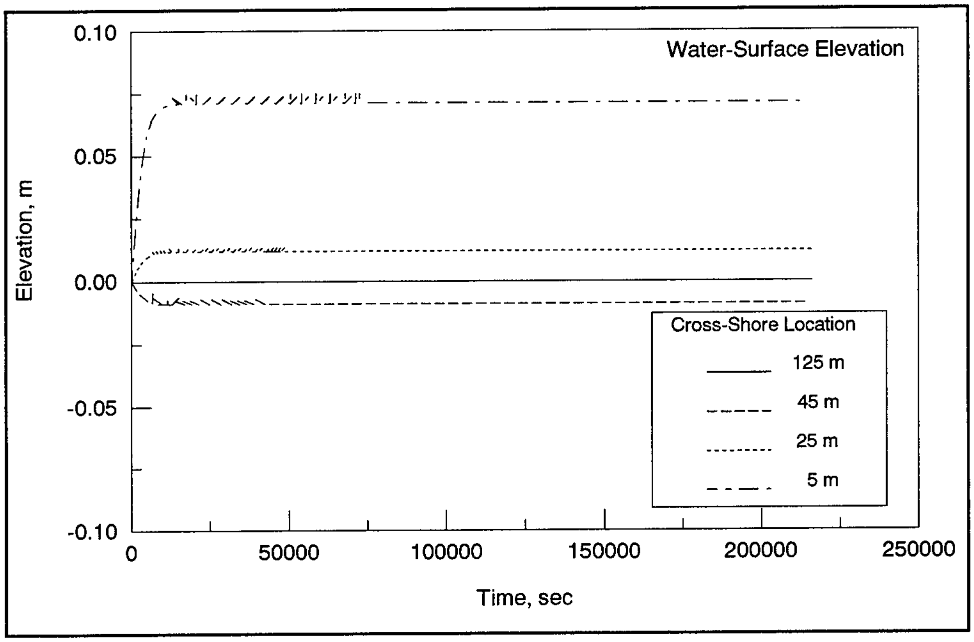

Figure 6-8. Calculated water-surface elevation versus time for Leadbetter Beach, California

Sediment Transport Model

To calculate sand transport and bottom change, a total-load sediment transport model was applied. The sediment transport model was embedded within ADCIRC for dynamic exchange of information between the two models. The sediment-transport algorithm applies the Ackers and White (1973) formulation for coarse material modified for the presence of waves (Bijker 1967). Bottom elevation change is given by

$$\frac{\partial D}{\partial t} = \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} \tag{6-10}$$

where

D = total depth relative to a datum

q_x = sediment transport directed along the x-axis

q_y = sediment transport directed along the y-axis

t = time

For detailed information on the methodology applied in the sediment transport calculations, see Scheffner (1996).

Coupling of the sediment-transport calculations with the hydrodynamics was conducted by calculating the change in bottom elevation over the grid. Sediment transport was calculated every 100th hydrodynamic time-step. After each set of sediment transport calculations over the grid, the bottom elevation was adjusted by the amount of material that entered or exited at each node. In this manner, time evolution of the bottom change was calculated. Figure 6-9 illustrates the coupling between the sediment transport and hydrodynamics.

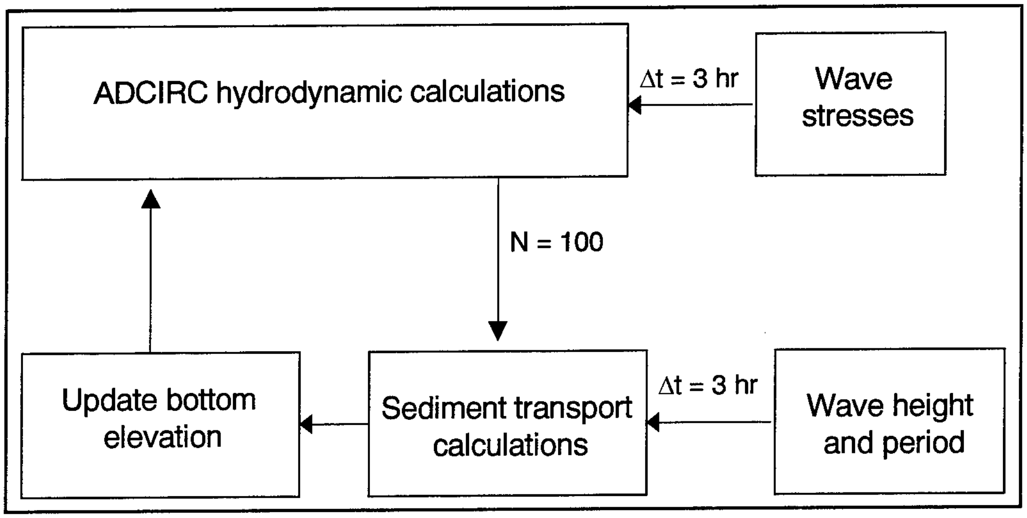

Figure 6-9. Flowchart of hydrodynamic and sediment transport calculations with wave parameter input

Because waves are a significant force for movement of sediment into the water column, all sediment-transport calculations were conducted with wave stress forcing included in the hydrodynamic (ADCIRC) simulations. Wave modeling was conducted prior to the sediment transport simulations and descriptions of the runs are given in Chapter 5. Wave fields (wave stress, height, and period) were updated every 3 hr in the hydrodynamic and sediment-transport calculations and held constant over each 3-hr interval. Wave calculations were conducted outside of the sediment-transport calculations, and wave stress, height, and period were not updated for changes in bottom elevation calculated by the sediment-transport model.

The ADCIRC grid domain for the Willapa Bay simulations is larger than the domain of the STWAVE grid. Wave parameters were calculated on the STWAVE grid and interpolated onto the ADCIRC grid. In areas where the two grids did not overlap, wave parameters were set to zero for sediment transport and wave stress forcing calculations. Thus, outside of the STWAVE grid domain, it was assumed that waves did not contribute significantly to sediment transport and wave-induced currents at Willapa Bay.

During initial testing of the combined hydrodynamic and sediment-transport modeling, stability was not maintained in the vicinity of the SR-105 structure because of strong advection. To enhance stability, advective terms in the momentum equations were turned off in all simulations in which sediment transport was coupled to hydrodynamics. Elimination of the advective terms did not significantly change the current patterns in the entrance, with the exception of areas adjacent to the SR-105 structure. Eddies that normally form next to the structure were not present in simulations without the advective terms. Sediment transport calculated without the advective terms in the velocity calculation (in the circulation model) is not expected to significantly alter the sediment transport values within the Alternative channels.

Verification of Sediment Transport Method

This section describes the performance of the coupled ADCIRC, STWAVE, and total-load sediment-transport calculations for idealized physical situations. The objective is to demonstrate reliability of the modeling system for an idealized situation prior to application at Willapa Bay.

The modified Ackers and White (1973) total-load formula as modified by Scheffner (1996) and implemented in ADCIRC was evaluated with bathymetry from the Leadbetter Beach, California, example described in the previous section. A sediment grain size of 0.3 mm was specified. Radiation-stress gradients were computed from STWAVE-calculated wave parameters for the three wave conditions shown in Table 6-4 for evaluating sediment transport rate predictions.

Table 6-4
Wave Parameters for Idealized Sediment Transport Cases

Case	H_s, m	T, sec	θ, deg
1	2	14	-10
2	1	12	-10
3	2	12	-15

The purpose of the exercise was to demonstrate that the total-load transport formula was properly implemented in ADCIRC and that the Ackers and White (1973) methodology was appropriate for surf zone conditions as occur on the entrance to Willapa Bay. Figure 6-10 shows the cross-shore variation of significant wave height for the three wave cases. In Cases 1 and 3, waves began breaking about 75 m from shore and decayed in approximately linear proportion to the decreasing depth across shore. In Case 2, waves shoaled slightly above 1.0 m then began to break about 45 m from the shore and decayed toward the shore.

Wave-driven currents were calculated by ADCIRC with wave-stress forcing. Three simulations were conducted that correspond to the wave parameters given in Table 6-4. Each ADCIRC simulation was conducted over a 12-hr interval in which the wave-stress field was held constant. Total-load calculations were computed at approximately Hour 4 of each simulation. Figure 6-11 shows the longshore current calculated for the three cases. Case 2 has significantly weaker current than Cases 1 and 3. This situation is expected because of the smaller wave height in Case 2. Figure 6-12 shows the corresponding longshore sediment-transport rate per unit width q_y, expressed in cubic meters per linear meter across shore per second. The variation in longshore transport rate corresponds to the cross-shore distribution of longshore current. The smallest longshore transport rate, with a peak value of 0.0005 m^3/m/sec, corresponds to Case 2 (1-m wave height). Even though Cases 1 and 3 have the same incident wave height of 2.0 m, the corresponding peak longshore transport rate for Case 1 (q_y = 0.0046 m^3/m/sec) is about half of that for Case 3 (q_y = 0.01 m^3/m/sec). The greater rate for Case 3 owes to the larger incident wave angle and shorter wave period.

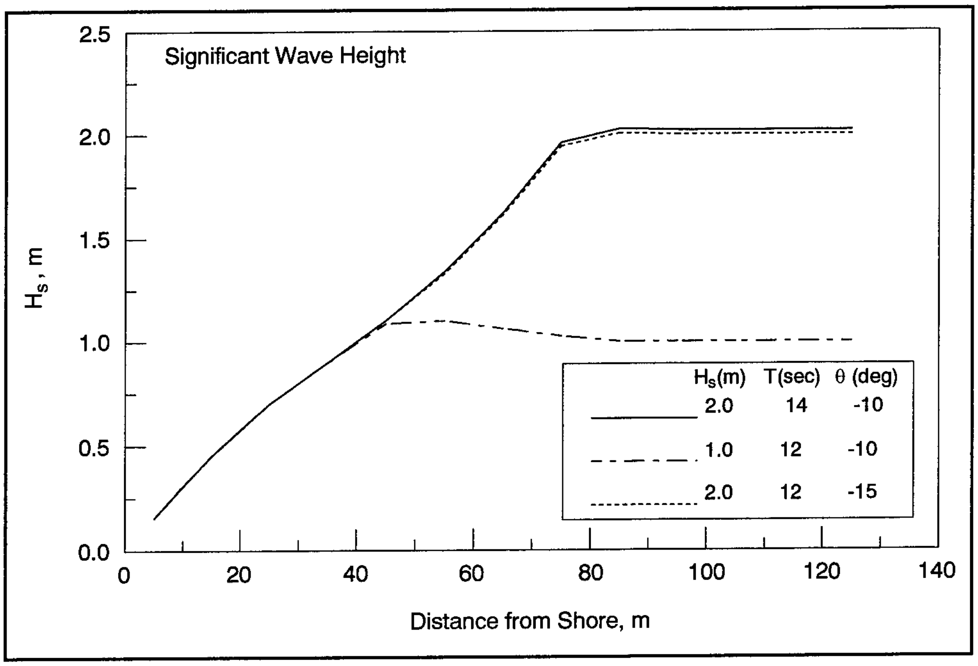

Figure 6-10. Cross-shore variation of significant wave height calculated by STWAVE

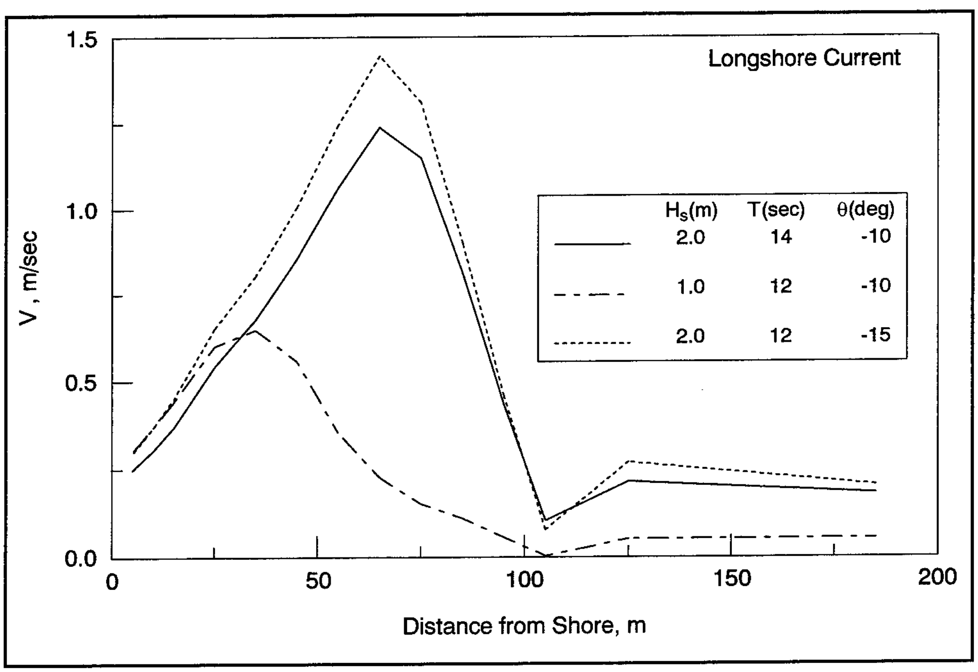

Figure 6-11. Cross-shore variation of longshore current calculated by ADCIRC

Figure 6-12 compares the Ackers and White total load calculation to the Coastal Engineering Research Center (CERC) formula (Shore Protection Manual 1984). The total longshore transport rate Q was calculated by integrating the transport rate per unit width q_y across the surf zone. The CERC formula, as a standard, has an expected accuracy of about ±50 percent (SPM 1984). The surf zone is defined as the area from the onset of breaking, calculated by STWAVE, to the beach. Two methods for calculating the total transport rate with the CERC formula are shown. The first method, labeled CERC/LIN, applies linear wave theory to calculate the wave parameters at the break point. The second method,

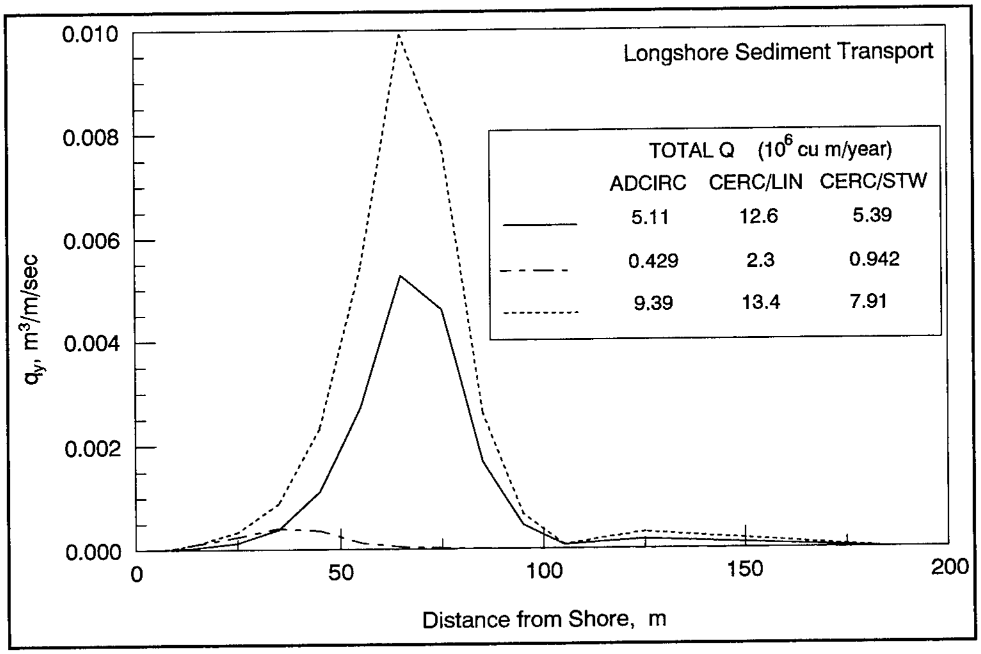

Figure 6-12. Cross-shore variation of sediment transport and total transport compared to the CERC formula for three wave cases

labeled CERC/STW, applies breaking wave parameters calculated by STWAVE. Both methods include the standard CERC formula for longshore transport rate given by:

$$Q = \frac{K}{16\left[\left(\rho_s/\rho\right)-1\right]a'}\sqrt{\frac{g}{\gamma}}\,\frac{H_b^{\frac{5}{2}}}{2.386}\sin\left(\theta_b\right) \tag{6-11}$$

where

K = nondimensional empirical sand transport coefficient (K = 0.77)

ρ_s = density of sediment (2.65 kg/m^3)

a' = volume of solids/total volume (accounts for sand porosity, a' = 0.6)

g = acceleration due to gravity

γ = wave breaking index ($H_s = \gamma\, d_b$, d_b = breaking depth, $\gamma = 0.78$)

H_b = breaking wave height, $\bullet_b$ = breaking wave angle.

Comparison of total longshore sediment-transport rates calculated by the Ackers and White formulation within ADCIRC and the CERC formula is given in terms of the ratio

$$R = \frac{Q_{AckWht}}{Q_{CERC}} \tag{6-12}$$

where Q_{AckWht} denotes the total longshore sediment-transport rate calculated by the Ackers and White formulation within ADCIRC, and Q_{CERC} denotes the total longshore sediment transport rate calculated by the CERC formula. For reference, if $R = 1$, then the transport rates computed within ADCIRC and the CERC formula are equal; if $R = 0.5$, then the transport rate calculated within ADCIRC is half that (50 percent below) calculated by the CERC formula.

Table 6-5 gives R values comparing sediment-transport rates calculated within ADCIRC and by the CERC formula. Sediment-transport rates calculated within ADCIRC compare well with those calculated by the CERC formula applying the CERC/STW method for the larger wave cases examined.

Table 6-5
Ratio of Sediment Transport Rates Calculated by ADTRANS and the CERC Formula

Case	*R*, ADCIRC and CERC/LIN	*R*, ADCIRC and CERC/STW
1	0.5	1.05
2	0.2	0.5
3	0.7	1.2

Salinity Model

Transport of salinity was calculated by the ADTRANS model, which applies the convection-diffusion equation to compute concentration of a material or substance. ADTRANS is linked to ADCIRC to provide coupled hydrodynamics and transport. The ADTRANS model can be applied to calculate concentration of conservative or nonconservative substances and requires specification of coefficients and boundary conditions appropriate for a given constituent. For example, calculation of salinity requires specification of representative salinity values at the ocean boundaries and at freshwater tributaries.

Concentration of a constituent is described within the ADTRANS model by the depth-averaged convection-diffusion equation written in nonconservative form as

$$D\frac{\partial C}{\partial t}+D\left[U\frac{\partial C}{\partial x}+V\frac{\partial C}{\partial y}\right]=-D\left[\frac{\partial}{\partial x}\left(K_x\frac{\partial C}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\frac{\partial C}{\partial y}\right)\right]+E-F \quad (6\text{-}13)$$

where

C = depth-averaged concentration of a constituent

K_x = horizontal diffusion coefficient applied for the x-directed concentration gradient

K_y = horizontal diffusion coefficient for the y-directed concentration gradient

E = source term

F = sink term

Source and sink terms are required for applications where modeled constituents can be produced or can decay within the system. Calculation of salinity does not require source or sink terms because salts are conservative substances.

Computational Grid

The computational grid developed for Willapa Bay encompasses a regional area extending from 40.8 to 51.2 deg North and from –130.5 to -122.7 deg West. Shoreline data were obtained from the NOAA Medium Resolution Shoreline Database, which is composed of shoreline position information digitized from NOAA navigation charts. Bathymetry data were obtained from several sources. National Ocean Service bathymetry data were applied in the Pacific Ocean. Within Willapa Bay, high-density bathymetry data were obtained from the Seattle District. These data were collected in 1998 as decribed in Chapter 3. Regions of Willapa Bay not covered by the 1998 surveys have bathymetry applied from NOS charts. Figure 6-13 shows bathymetry contours of Willapa Bay calculated from the composite data, and Figure 6-14 shows detail of bathymetry in the entrance. Tidal-flat elevations were estimated from NOS charts. Typical tidal-flat elevation specified in the grid is 1.5 m below mtl.

The computational grid contains 24,170 nodes and 46,250 elements. Lowest resolution was specified in the Pacific Ocean, and highest resolution was specified in the entrance to Willapa Bay. Figure 6-15 shows a regional view of the grid that illustrates the variation in resolution over a portion of the domain. Figure 6-16 shows the grid for the entire Willapa Bay, and Figure 6-17 shows grid detail in the entrance. Resolution for the Alternative channels can be seen in Figure 6-17. Alternatives represented in this grid are 1, 3A, 3B, 3F, 3G, 4A, and 4E. High resolution was specified so that direct comparisons of velocity and flow rate can be made among Alternatives. Figure 6-17 shows the base grid for the existing condition based on 1998 bathymetry (Figure 6-13), and bathymetry has not been modified for representation of action Alternatives. Alternative 1 (no action) is shown with a dashed line to separate it from action Alternatives.

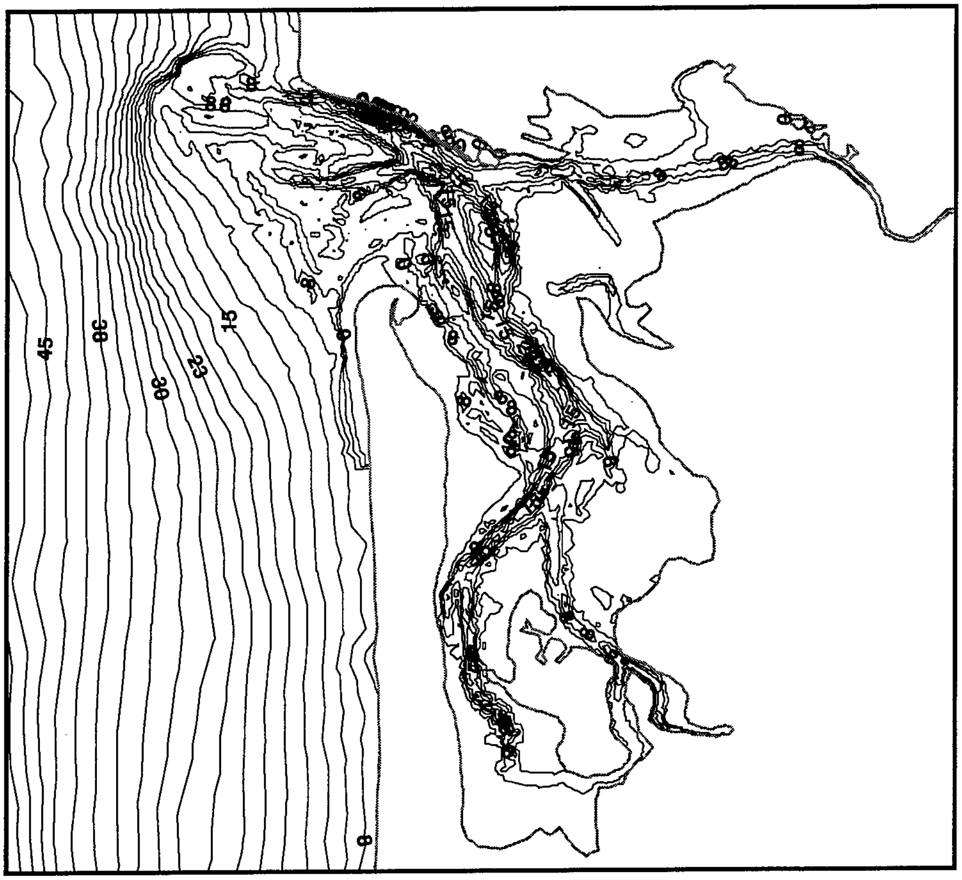

Figure 6-13. Willapa Bay bathymetry. Contours at 2.5-m intervals relative to mtl

Figure 6-14. Willapa Bay entrance bathymetry

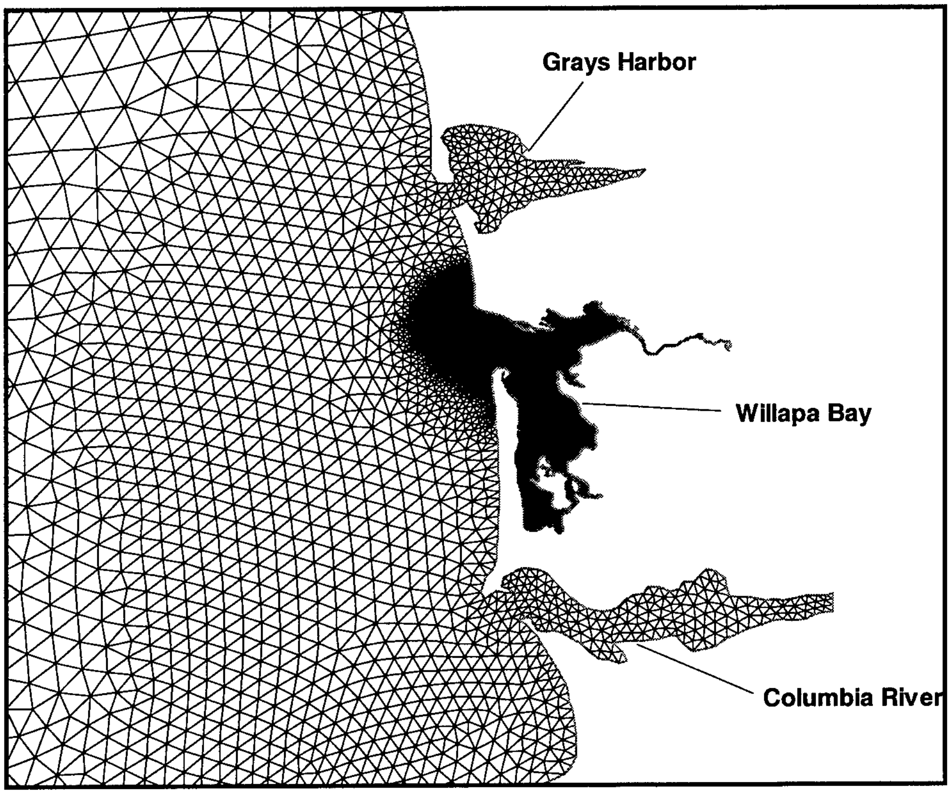

Figure 6-15. Regional view of grid for coast of Washington and Oregon

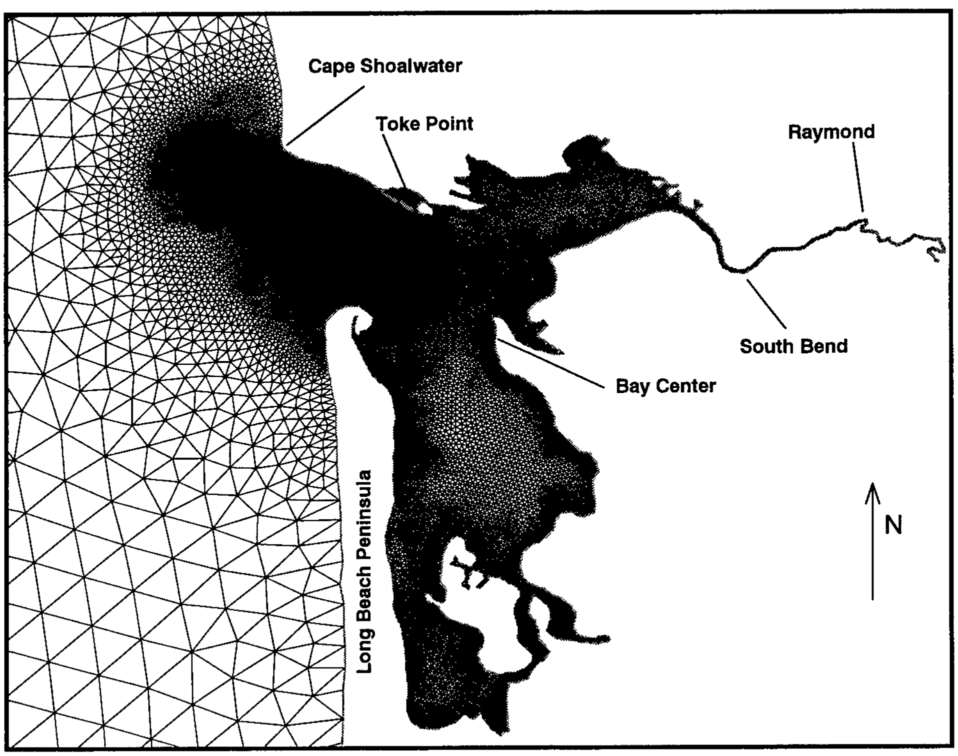

Figure 6-16. Grid detail for Willapa Bay

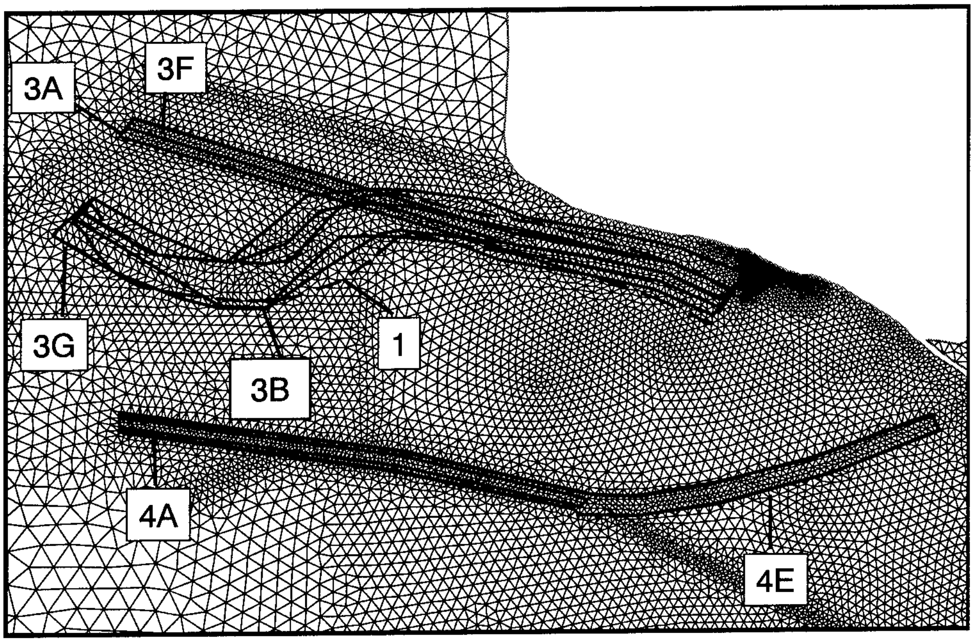

Figure 6-17. Grid detail in Willapa Bay entrance (existing bathymetry with resolution for Alternatives with the exception of Alternatives 3H-a and 3H-b)

For each Alternative requiring dredging, a separate grid was developed from the base grid where modifications (dredged channel and/or structures) were implemented. Changes in the grid for action Alternatives (those requiring dredging) involved only modification to bathymetry; resolution remained consistent with the base grid, with the exception of Alternatives 3H-a and 3H-b (described in the following paragraph). Figure 6-18 shows nodes (calculation points) at the entrance to Willapa Bay. In locations where Alternative channels were proposed, node density in the base grid was increased above that in surrounding areas. In general, channels for Alternatives were specified as 500 ft and 1,000 ft in width. Representation of these channels was accomplished by placing three nodes across 500-ft channels and five nodes across 1,000-ft-wide channels. Nodes for the 500-ft-wide channels overlap those in the 1,000-ft-wide channels. Because of the high resolution in the entrance, the model was run with a 1.5-sec time-step to maintain numerical stability. Contour plots of bathymetry for each Alternative are shown in Appendix G.

Modifications to the base grid for Alternatives 3H-a and 3H-b were required because these Alternatives were defined after the base grid was developed. Alternative 3H-a consists of a 28-ft channel dredged along a line extending from the end of the SR-105 groin and dike seaward across the bar at a direction of approximately 285 deg relative to True North. Alternative 3H-b has the same configuration as Alternative 3H-a except for an increase in the dike crest elevation from -18 to -2 ft mllw. Changes implemented for Alternatives 3H-a and 3H-b included increased resolution along the channel alignment as specified previously for the other Alternatives, as shown in Figures 6-19 and 6-20. The revised grid contains 46,970 elements and 24,530 nodes. Alternatives 3H-a and 3H-b were the only channel configurations simulated with the revised grid.

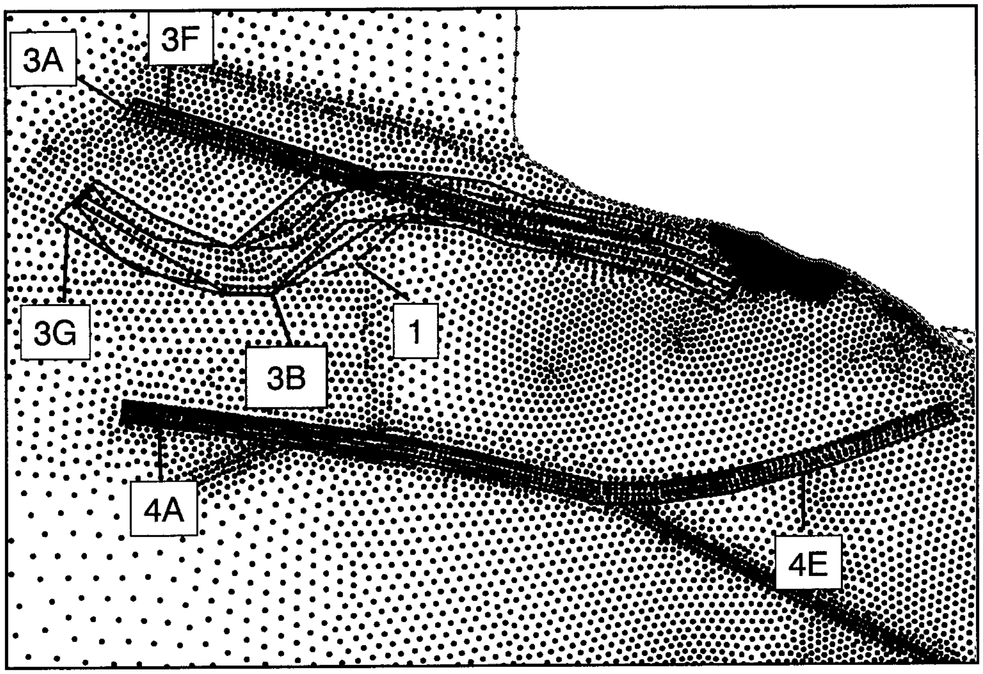

Figure 6-18. Nodes in Willapa Bay entrance (existing bathymetry with resolution for Alternatives with the exception of Alternatives 3H-a and 3H-b)

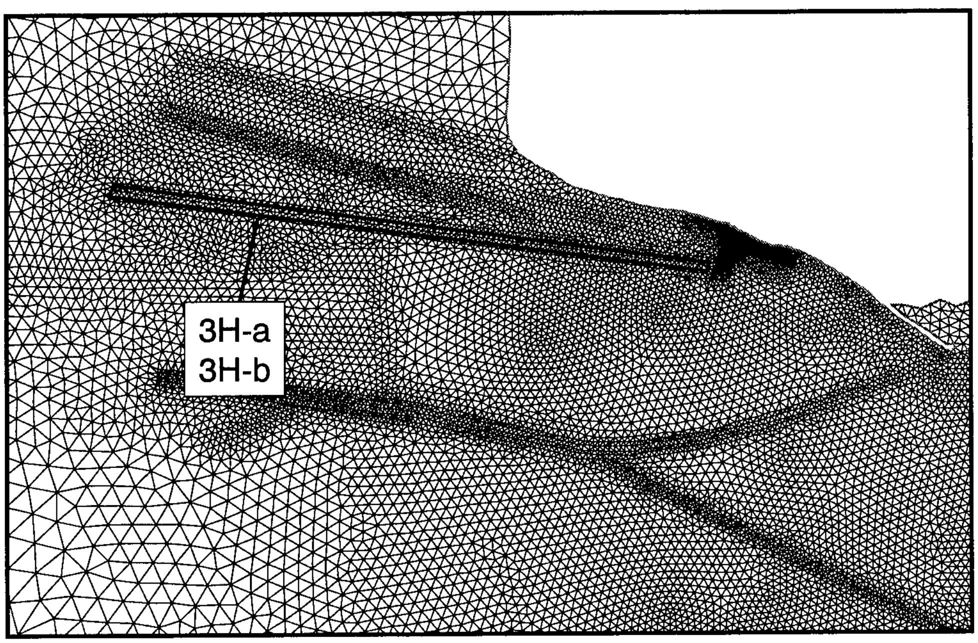

Figure 6-19. Grid detail in Willapa Bay entrance with resolution for Alternatives 3H-a and 3H-b

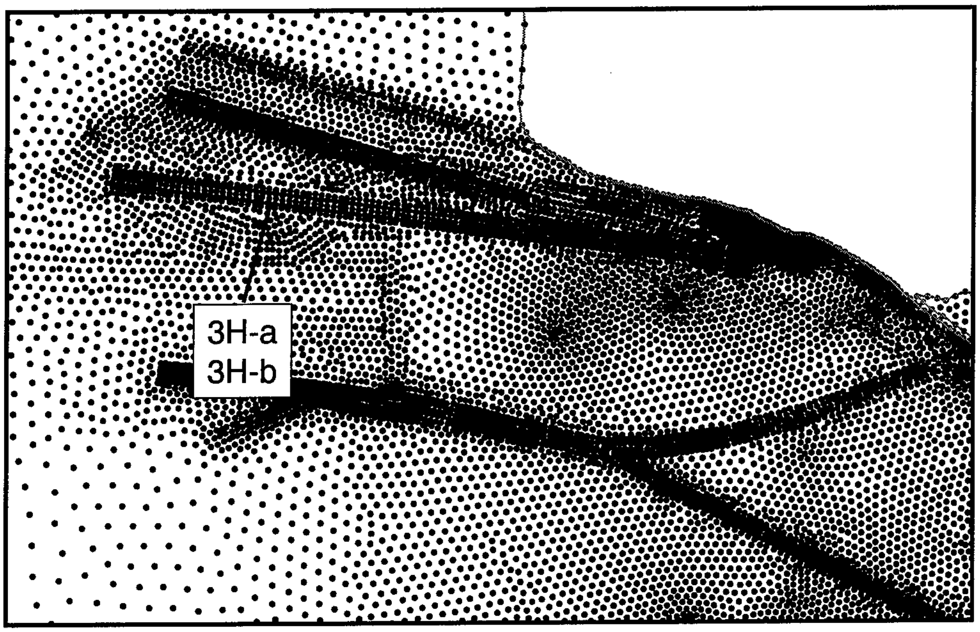

Figure 6-20. Nodes in Willapa Bay entrance with resolution for Alternatives 3H-a and 3H-b

To enhance numerical stability, the crest elevation of the dike was raised to +9.8 ft msl. Because the dike elevation in the Alternative 3H-b grid is higher than that specified for this Alternative, changes in calculated velocity for Alternative 3H-b over those of Alternative 3H-a are expected to be greater than what would occur if the dike were raised to the design elevation of -2 ft mllw. However, velocity changes caused by this structural modification are assumed to be qualitatively correct.

The dike-and-groin structure located in the North Channel was included in the grids. Pacific International Engineering provided information on the structure including its position, dimensions, and crest elevations. Representation of the structure in the base grid is shown in Figure 6-21 in terms of nodes. An image overlay of the structure is shown for reference. Nodes were specified along the crest and foot of the structure. Grid resolution at the SR-105 groin and dike was increased for Alternatives 3H-a and 3H-b to better resolve local velocity changes.

Circulation Model Calibration

Calibration of the circulation model (ADCIRC) was conducted for tidal conditions. Initial calculations by the circulation model overpredicted water level and current speed at four measurement stations, shown in Figure 6-22, at Willapa Bay. Examination of the overpredictions revealed that the M_2 tidal-constituent forcing amplitude was too large. This amplitude was reduced by 20 cm at all ocean forcing boundary nodes, which led to good agreement between the measurements and model. The tidal constituent database (Le Provost et al. 1994) from which the tidal parameters were obtained was not verified for much of the Pacific Ocean region where the Willapa Bay model was forced. Because the tidal constituents were calculated from a numerical model, errors are present

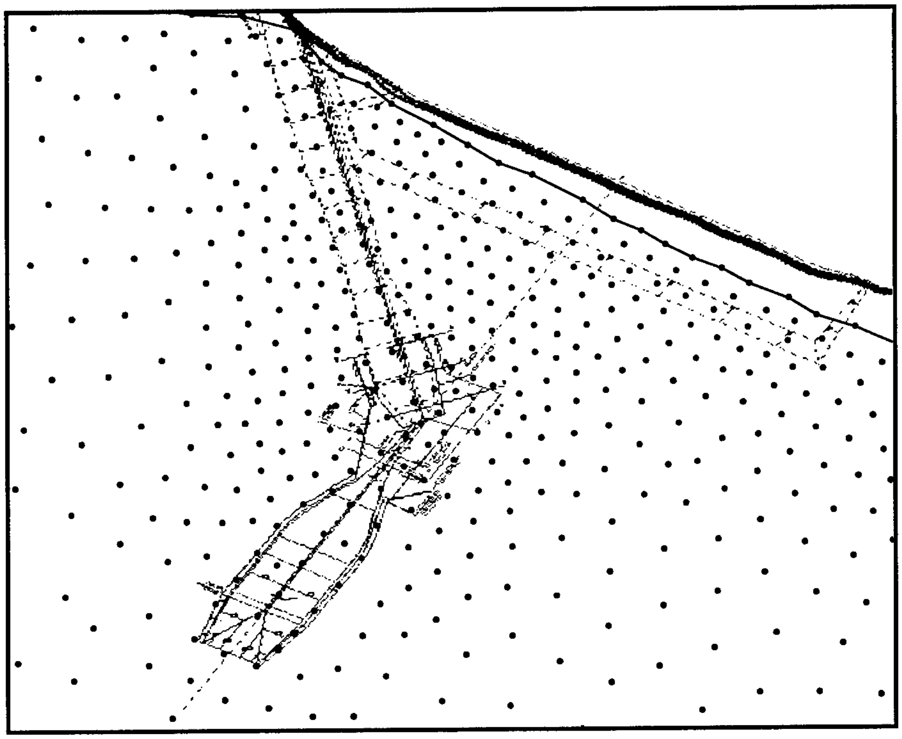

Figure 6-21. Nodes at SR-105 dike and groin

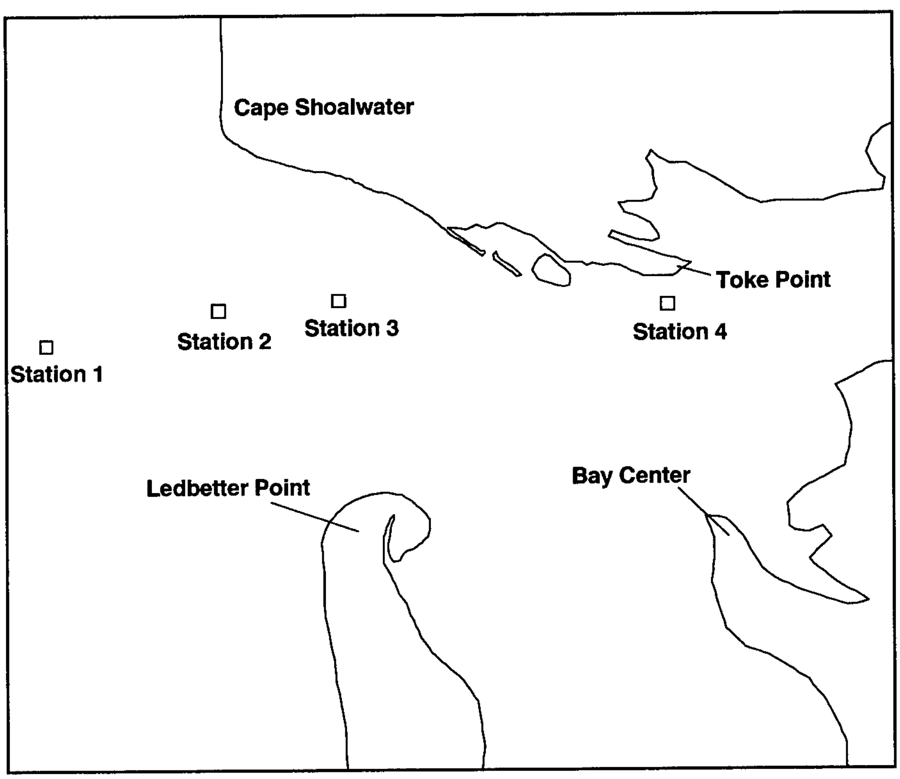

Figure 6-22. Locations of stations for comparisons of water-surface elevation and current

in the database. Therefore, amplitudes and phases may be adjusted for increased accuracy in simulations. For this project, the M_2 amplitude reduction ranged from a 14 to 19 percent change from values extracted from the database.

The friction factor was set to 0.0025 throughout the model domain and was not adjusted in the calibration process.

Circulation Model Verification

Comparisons between calculated and measured water level and current velocities are presented for a 1-month tidal simulation for the 1998 bathymetry. Water level and velocity were compared for the four stations shown in Figure 6-22. Model forcing was specified with tidal constituents only; wind and freshwater influx were not applied. The time interval selected for comparison commenced on 5 September 1998 (Day 248) and ended on 6 October 1998 (Day 279). This time interval was selected for initial comparison because data indicate that there was little influence of the wind or other nontidal processes.

Comparison of water-surface elevation

Calculated water-surface elevation at the four measurement stations is compared with measurements and described. Water-level measurements were de-meaned (mean subtracted) for comparison with calculated values. Because the calculations were conducted with only diurnal and semidiurnal tidal forcing (no wind or river discharge), the measurements were high-pass filtered with a cutoff frequency of 0.6667 cycles per day (cpd) to remove motion with duration longer than 1.5 days. The calculated water levels at the four stations were in good agreement with measurements. Peaks and troughs of the calculated tidal wave were in phase with those of the measurements. Figures 6-23 through 6-26 plot calculated and measured water level at Stations 1 through 4, respectively. Calculated water levels exhibit increased tidal range with distance into the bay, which is a property of the Willapa Bay system.

Error in calculation of water level was quantified by computing rms error and percent error between measured and calculated water level at Stations 1 through 4. The rms error E_{rms} of a variable χ is

$$E_{rms} = \sqrt{\frac{\sum_{i=1}^{N}\left(\chi_{meas,i} - \chi_{calc,i}\right)^2}{N}} \tag{6-14}$$

where

$\chi_{meas,i}$ = i^{th} measured value of χ

$\chi_{calc,i}$ = i^{th} calculated value of χ

N = is the number of points.

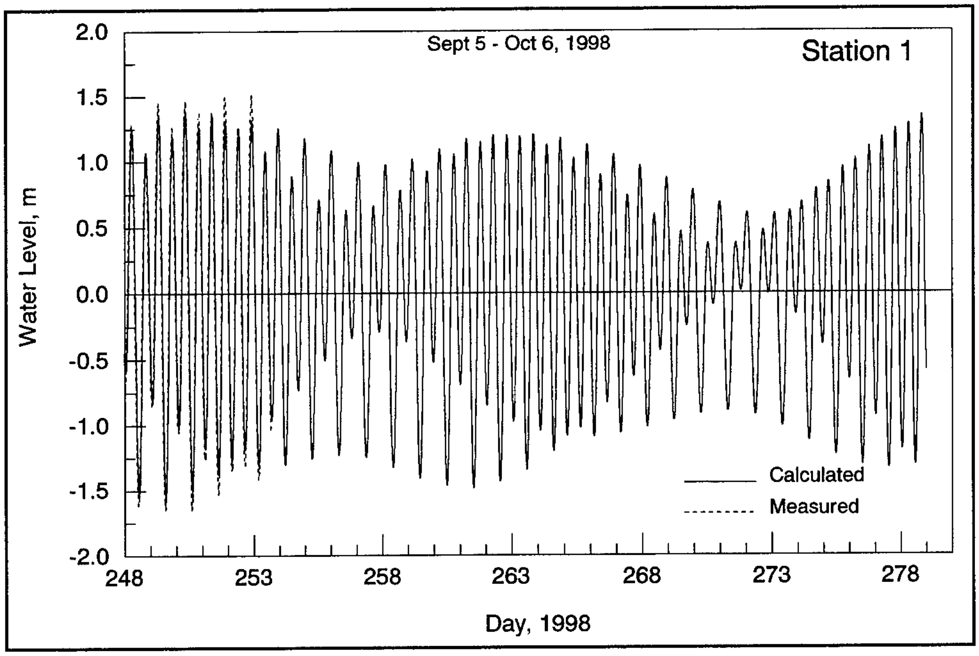

Figure 6-23. Time series of calculated and measured water level at Station 1

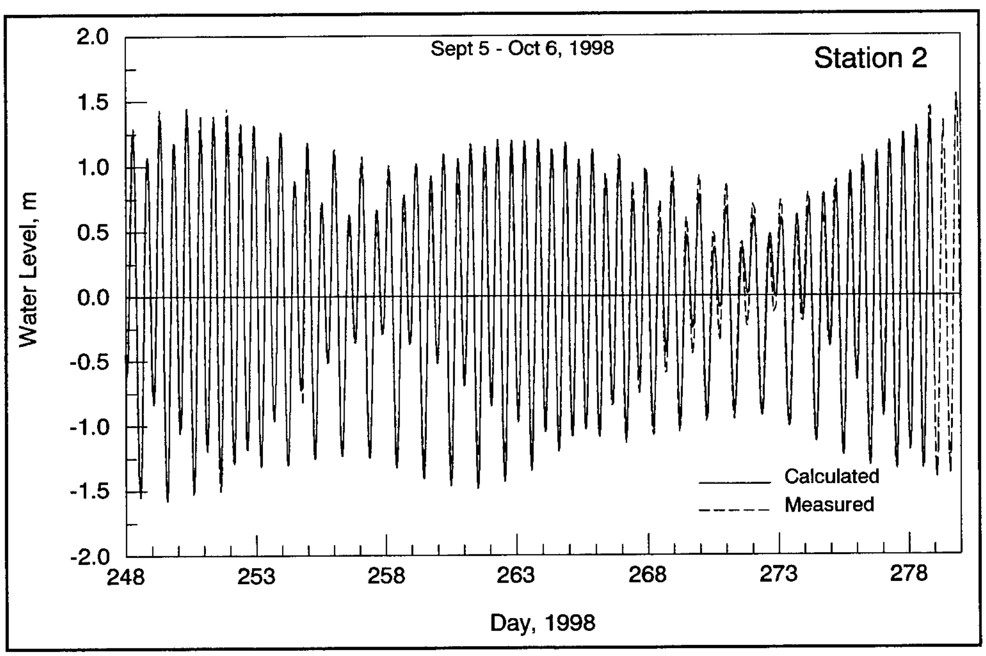

Figure 6-24. Time series of calculated and measured water level at Station 2

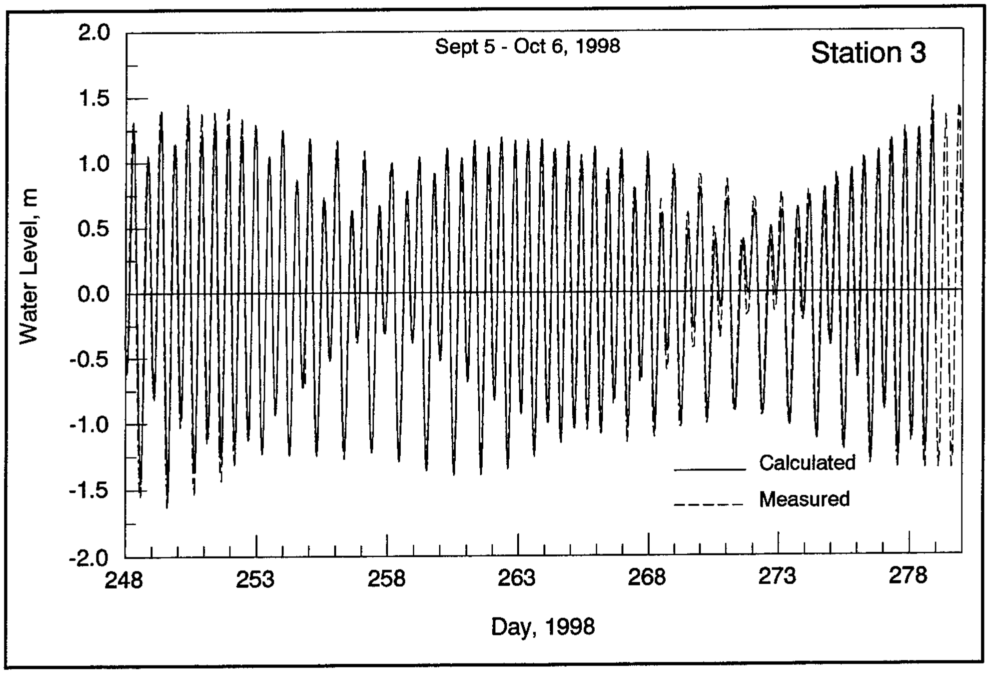

Figure 6-25. Time series of calculated and measured water level at Station 3

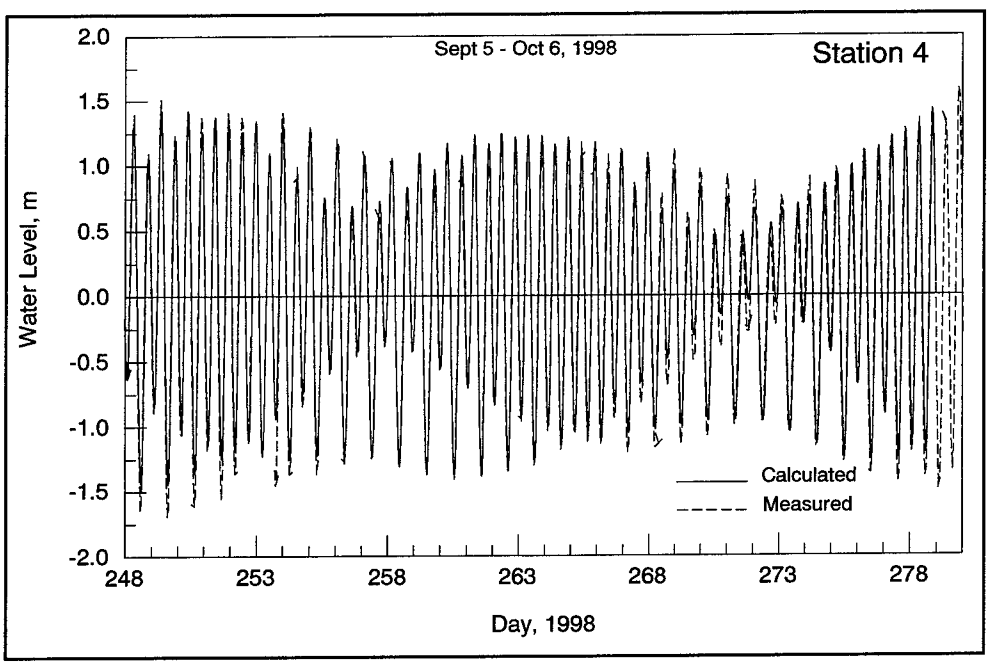

Figure 6-26. Time series of calculated and measured water level at Station 4

Percent error E_{PCT} in water level was calculated by

$$E_{PCT} = \frac{E_{rms}}{R_t} \tag{6-15}$$

where R_t is the tide range at Toke Point, taken to be the difference between the mhhw and mllw datums given in Table 6-2. Toke Point was selected as the reference point for the comparisons because it was the closest location to the measurement stations at which tidal datums were available. Table 6-6 gives the rms error and percent error for Stations 1 through 4 for the 30-day verification of the base grid. Because of gaps in measurements at Stations 3 and 4, the time series applied for error calculations was shorter than for Stations 1 and 2. Data gaps during the verification period are listed in Table 6-6. Calculated water levels were in good agreement with measurements. Minimum and maximum percent errors were 6 and 9 percent at Stations 1 and 3, respectively, for the base grid. Error calculations were also conducted for the grid that was modified for resolution of the Alternative 3H-a and 3H-b channels. The percent errors for the modified grid were within 1 percent of those for the base grid, except at Station 2 where it was within 2 percent. Thus, the increased resolution of the modified grid did not significantly alter the calculated values.

Table 6-6
Error in Calculated Water Level

Station	rms Error, m	Percent Error	Day Range, 1998	Data Gap, Days 1998
1	0.15	6	247.75 – 253.67	253.71 – 278.96
2	0.19	7	247.75 – 278.96	252.71 -- 254.75
3	0.25	9	247.75 – 278.96	252.71 – 254.78
4	0.18	7	247.75 – 278.96	247.79 – 253.68

Comparison of velocity

Measured and calculated velocities were compared at the four stations shown in Figure 6-22 for the 1-month verification simulation. Velocities were decomposed into north-south and east-west components. Decomposition of velocity provides information on dominant flow direction and agreement between measured and calculated values along specific directions. Measured velocities were depth averaged and band-pass filtered for the comparisons. Band-pass filtering was performed to remove nontidal motion from the velocity measurements because the verification simulation was conducted for tidal forcing only. The low- and high-frequency cutoffs for the filter were 0.6667 cpd and 8.0 cpd, respectively. All time-series plots of measured and calculated currents show the band-pass filtered measurements.

Velocity comparison for Station 1 is shown in Figure 6-27. Direction of flow is denoted in the plot. Overall, the model reproduces the measurements for the east-west velocity component, although it tends to underpredict the peak velocity during Days 248 through 254 and 275 through 278 and overpredicts during Days 261 through 268. For the north-south flow component, the model agrees

overall with the south-directed flow peaks, but underpredicts the northward-directed flow peaks during the same intervals as when the east-west velocity is underpredicted.

Comparison of velocities at Station 2 is shown in Figure 6-28. The magnitude and direction of the calculated velocity are in good agreement with measurements. The dominant east-west flow at Station 2 is reproduced in the calculations. Peak east velocity is underpredicted by a typical value of 0.25 m/sec. The north-south current component is weak at Stations 2, 3, and 4 compared with the east-west component. Scales on the y-axes of the velocity plots should be noted for comparison of the east-west and north-south currents.

Currents at Station 3 are plotted in Figure 6-29. As at Station 2, calculations agree well with measurements, both in magnitude and phase. The dominant east-west flow is reproduced in the calculations. Typical calculated eastward current is underpredicted by approximately 0.2 m/sec.

Currents at Station 4 are shown in Figure 6-30. The east-west current component shows overall good agreement throughout the simulation, although the calculated current consistently overpredicts the measured current by approximately 0.4 m/sec on the east-directed flow. The north-south component is weak, with the calculations apparently out of phase with measurements. This phase difference is attributed to relatively small-scale bathymetric features that are not resolved in the model. In addition, the range of the north-south velocity component is smaller after Day 253 than it is before Day 253. This change in range occurred when the instrument was serviced and the mount redeployed. The location of the mount was different before and after servicing, which explains the difference in the north-south current component.

Table 6-7 lists error statistics between measured and calculated velocity components at the measurement stations. Error was calculated with the band-pass-filtered measurements. Because datum-like values are not available for velocity, percent error was calculated with the range of values (maximum – minimum) contained in the measurements as R_t in Equation 6-15. Percent error for the east-west velocity component was less than 10 percent for Stations 1, 2, and 3 and was 12 percent for Station 4. These low errors indicate good agreement between calculated and measured velocity. The percent error for the north-south component of velocity is greater, with values of 14 and 13 percent for Stations 1, 2, and 3, and a value of 29 percent for Station 4. Stations 2, 3, and 4 have relatively weak north-south velocity components, and a small-magnitude error can give a large percent error. For Station 4, the large percent error owes primarily to the phase difference between the measured and calculated velocity.

Circulation patterns

Representative circulation patterns for the flood and ebb portions of the tidal cycle are presented in the form of vector plots. The vectors have been plotted with uniform distance between them rather than at each calculation point. This distribution of vectors increases readability of the plots, particularly in regions dense with calculation points.

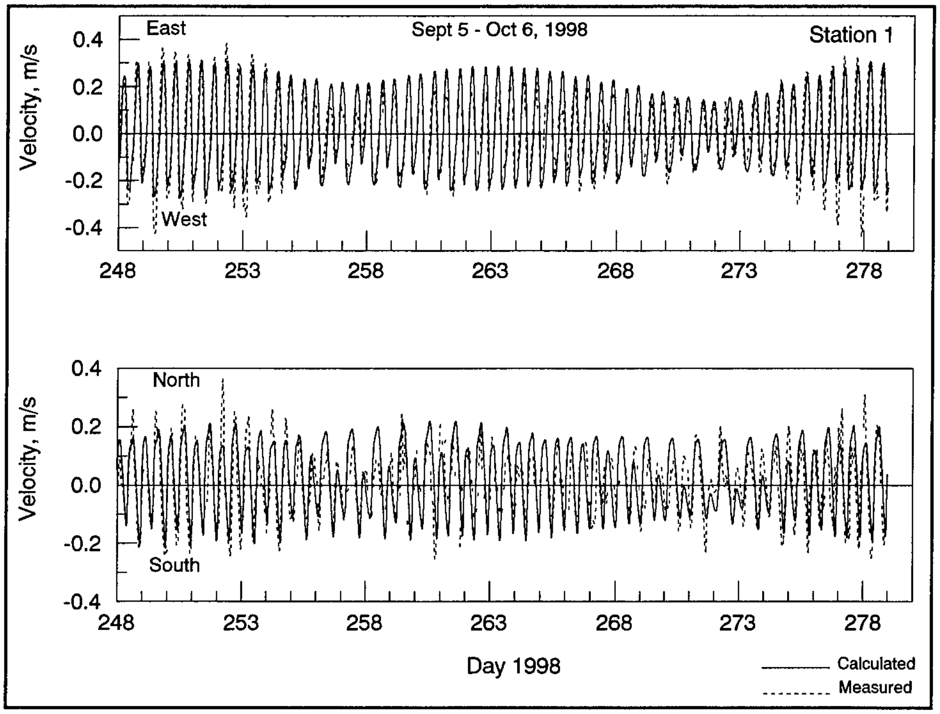

Figure 6-27. Time series of calculated and measured (band-pass filtered) current at Station 1

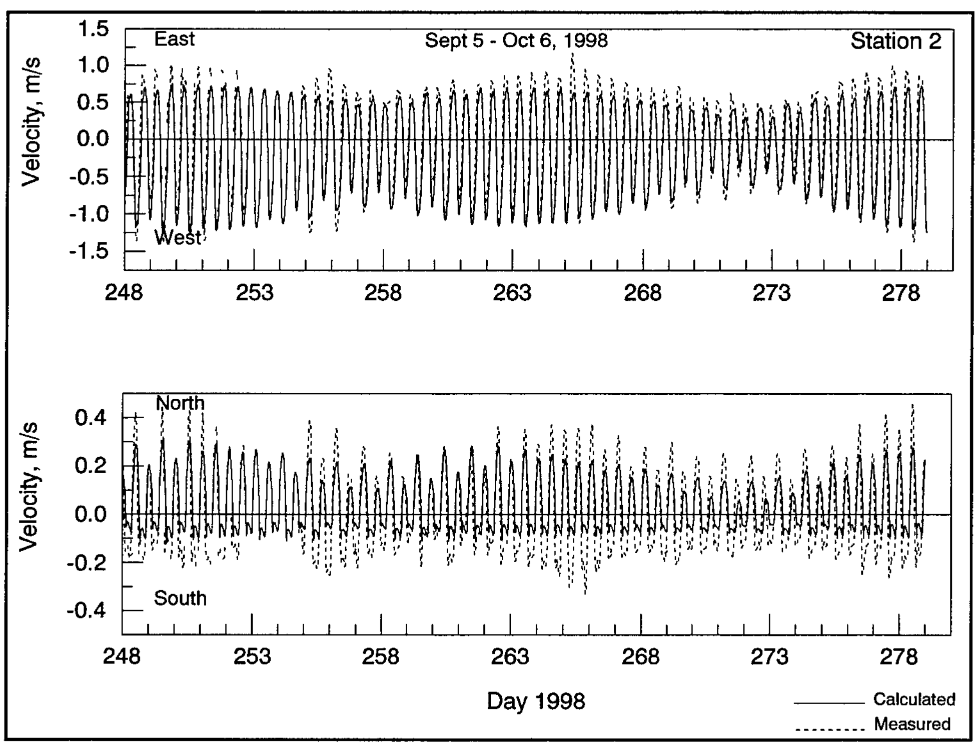

Figure 6-28. Time series of calculated and measured (band-pass filtered) velocity at Station 2

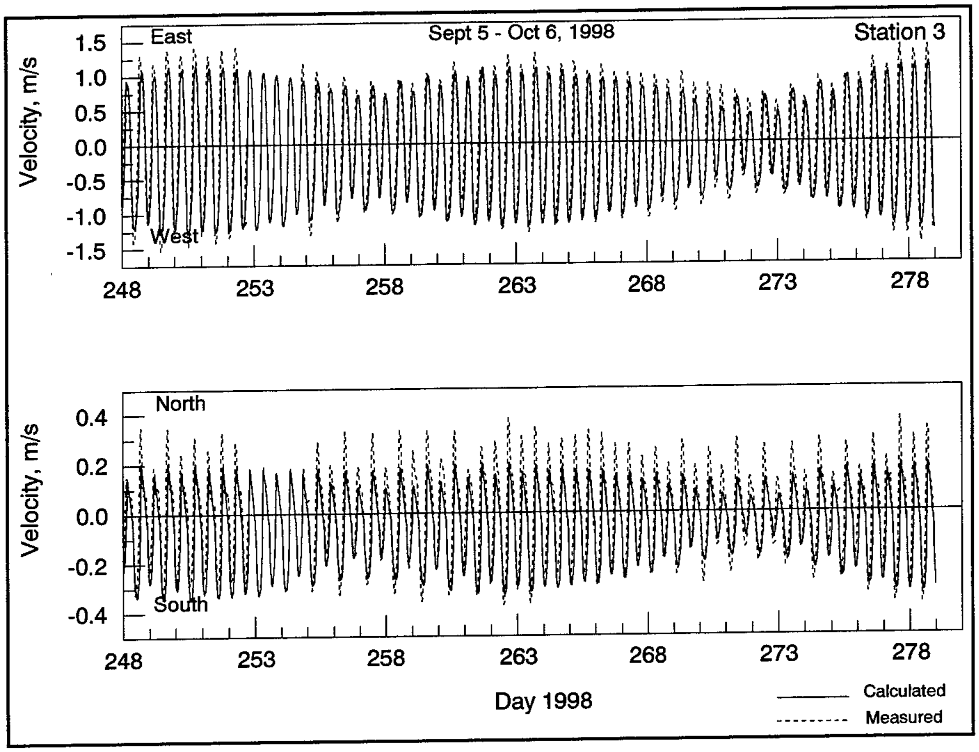

Figure 6-29. Time series of calculated and measured (band-pass filtered) velocity at Station 3

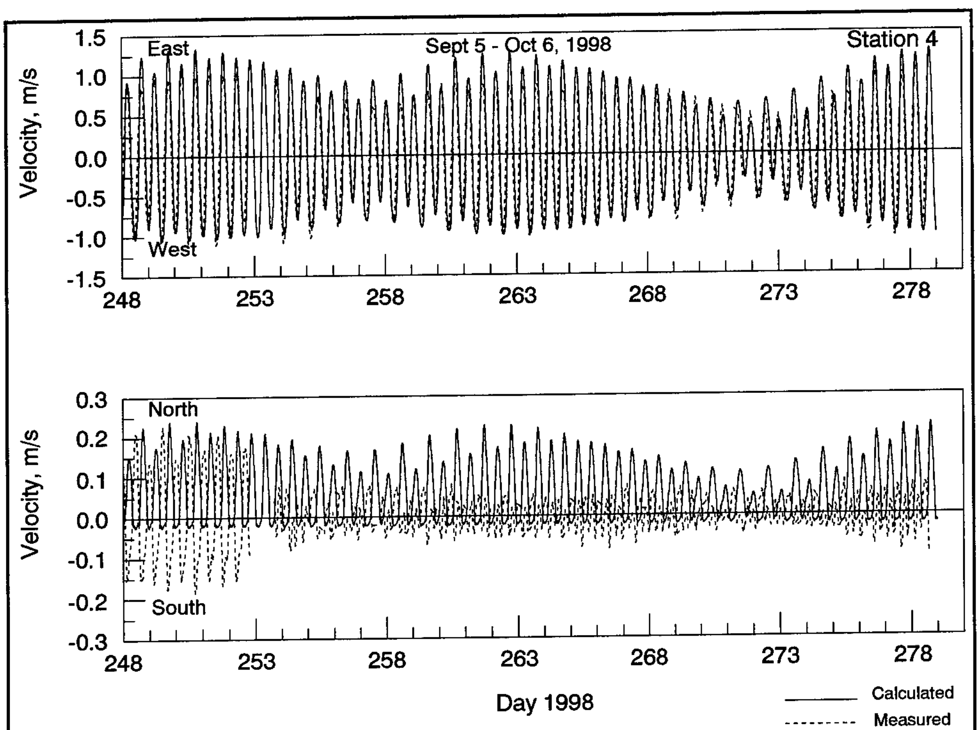

Figure 6-30. Time series of calculated and measured (band-pass filtered) velocity at Station 4

Table 6-7
Error in Calculated Velocity Components

Station	East – West rms Error, m/sec	East – West Percent Error	North – South rms Error, m/sec	North – South Percent Error	Day Range, 1998	Data Gap, Days 1998
1	0.07	8	0.09	14	248.01 – 278.98	Numerous[1]
2	0.23	9	0.11	14	248.01 – 278.98	Numerous, limited range[2] 252.70 – 254.77
3	0.27	9	0.11	13	248.01 – 278.98	252.73 – 254.82
4	0.27	12	0.12	29	248.00 – 278.67	252.77 – 253.70

[1] Brief gaps are present throughout data set; gaps range from 1 to 4 points.
[2] Brief gaps are present through time interval 248.16 to 252.60; gaps range from 1 to 4 points.

Figures 6-31 and 6-32 show circulation patterns for Willapa Bay on flood and ebb tide, respectively, where no wave or wind forcing has been imposed. During the flood cycle, strong flows are seen in the entrance and main bay channel. Channelized flow is evident in the southern portion of the bay and into the Willapa River. During ebb, strong flow occurs in the major channels similar to the flood tide, but in the opposite direction. Strong flows are present in the entrance, particularly in the North Channel. Over the bar, the northern portion of the flow (toward the northwest) has stronger velocities further from the entrance than do the central and southern portions of the flow (directed toward the west and southwest). This stronger northern flow appears to be forced primarily by the current in the North Channel, with some contribution from flow moving over the central region of the bar.

Calculated tidal-circulation patterns in the entrance to Willapa Bay on flood and ebb tide are shown in Figures 6-33 and 6-34, respectively. Flow in the entrance can be roughly divided into two portions, that through the North Channel and that over the shallower bar region south of the channel. In both figures, strong channelized flow is present in the North Channel. The bar creates a region of converging flow on flood tide and diverging flow on the ebb cycle. Current patterns are well organized during both ebb and flood tide.

During storms, waves can significantly modify the current patterns from those of the tide alone. Figures 6-35 and 6-36 show calculated currents in the entrance to Willapa Bay during the January 1998 storm at flood and ebb tide, respectively. Wave and wind forcing were applied in the storm calculations. During the storm, currents at the entrance exhibit complex patterns as a result of wave transformation and breaking.

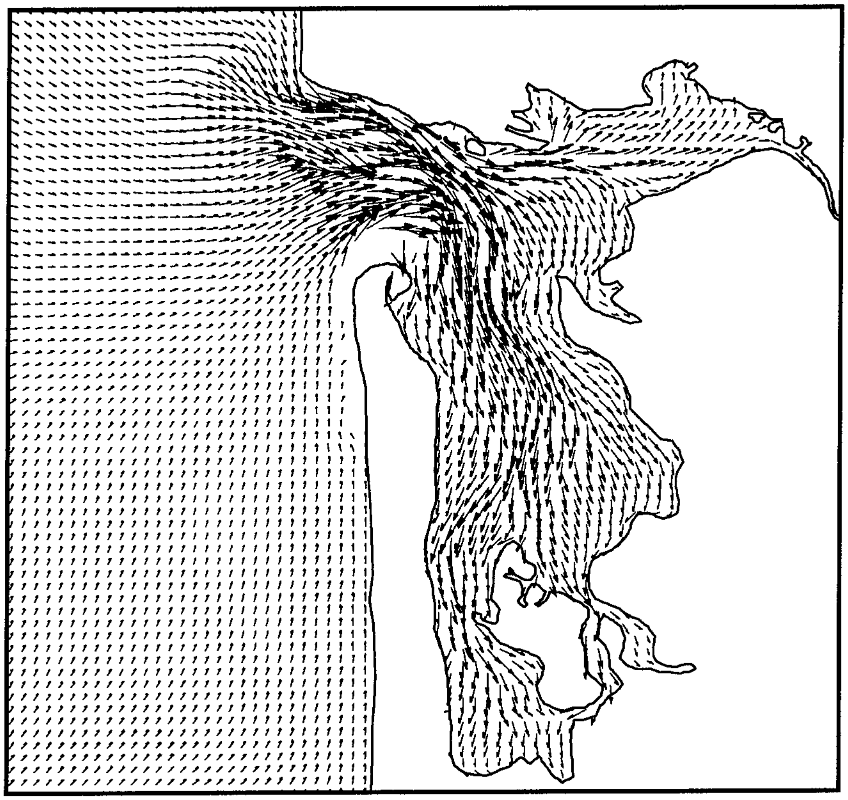

Figure 6-31. Willapa Bay circulation pattern on flood tide

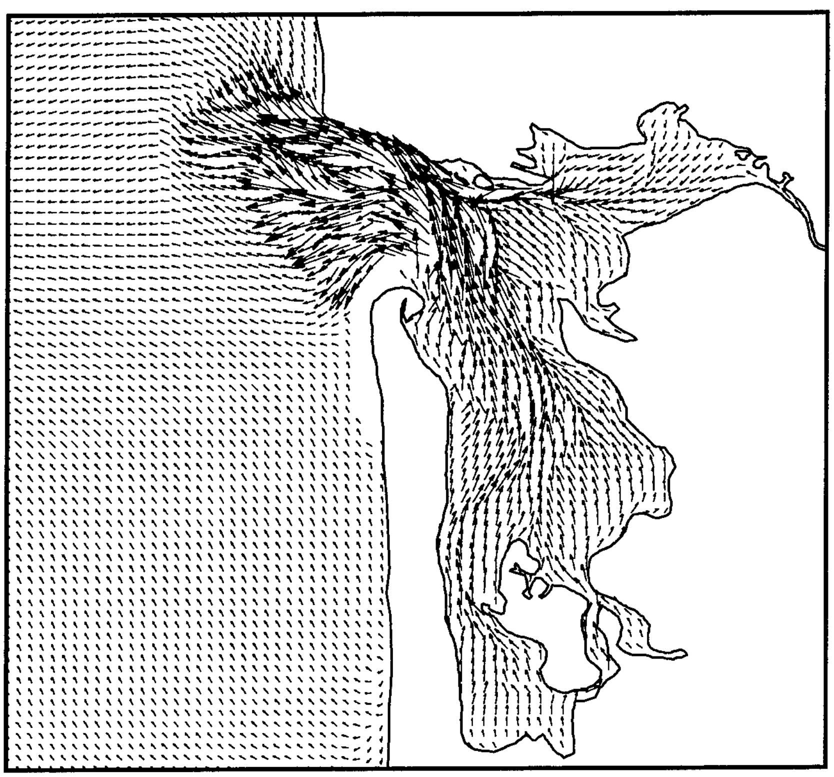

Figure 6-32. Willapa Bay circulation pattern on ebb tide

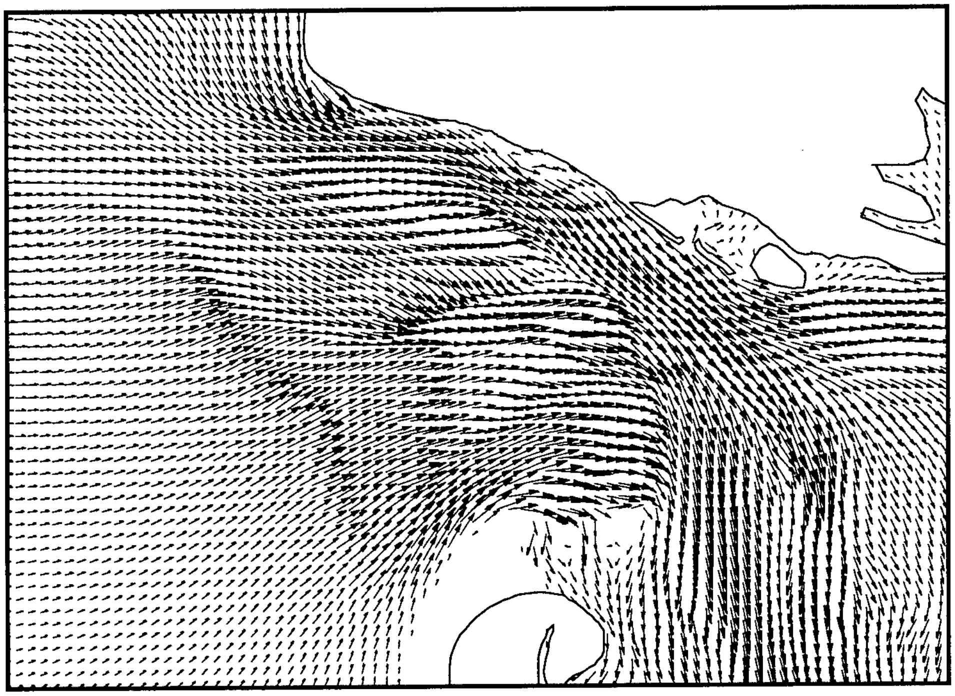

Figure 6-33. Entrance of Willapa Bay circulation pattern on flood tide

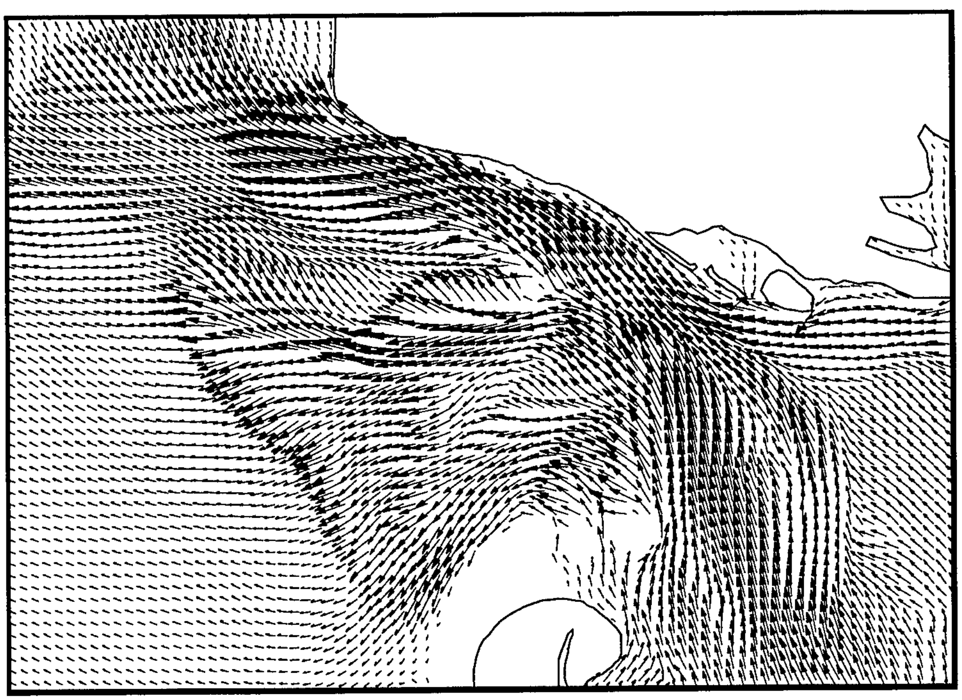

Figure 6-34. Entrance of Willapa Bay circulation pattern on ebb tide

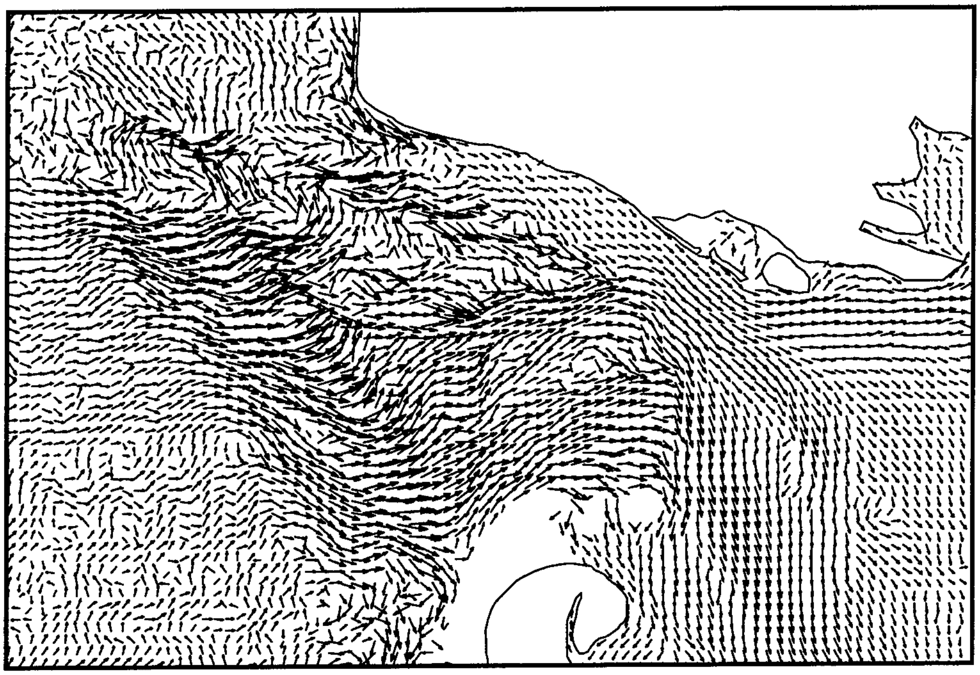

Figure 6-35. Entrance of Willapa Bay circulation pattern on flood tide during January 1998 storm

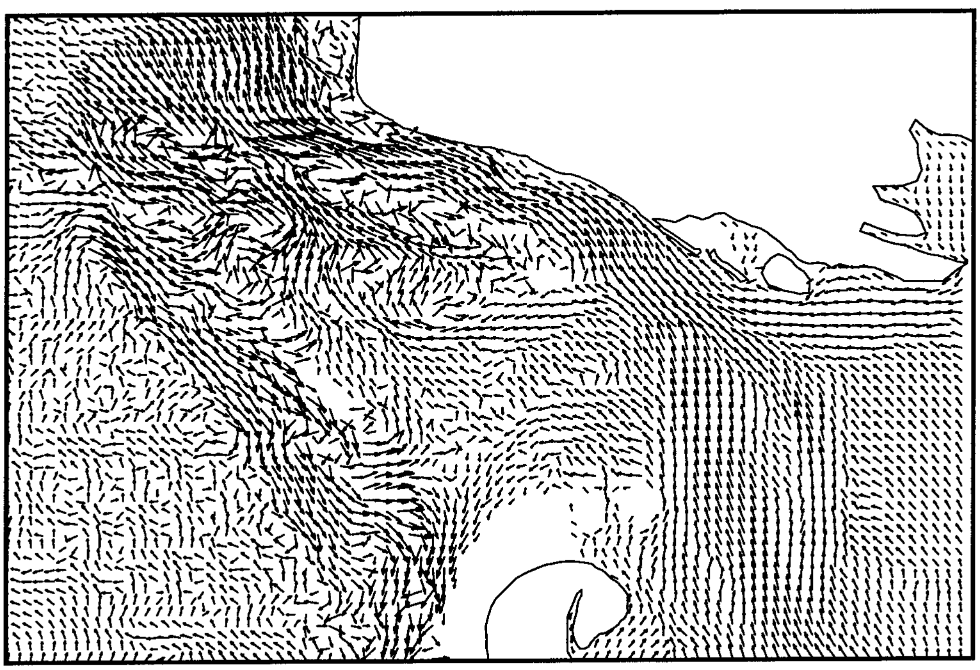

Figure 6-36. Entrance of Willapa Bay circulation pattern on ebb tide during January 1998 storm

Flow around the SR-105 dike and groin structure during flood and ebb tide is shown in Figures 6-37 and 6-38, respectively, for the situation of only tidal forcing. The above-water portion of the structure is shown in the figures where no vectors are present in the flow. On the flood cycle (Figure 6-37) an eddy forms upstream of the structure. The eddy circulation results in a northwest-directed current in a limited region located east of the structure. During the ebb cycle (Figure 6-38), an eddy is formed on the west side of the structure creating an eastward flow north of the eddy. On both flood and ebb cycles, velocities east and west of the structure are weaker than those in the North Channel. Note that on both cycles, flow is directed east on the west side of the structure, whereas it is directed toward the west on the eastern side of the structure.

Evaluation of Design Alternatives by Analysis of Hydrodynamics and Sediment Movement

Calculated current velocity and sediment transport are evaluated for Alternative channels. The model results provide guidance as to where areas of channel infilling can be expected and identify reaches of channels where navigation would be difficult or hazardous. By this means, Alternatives can be compared on an objective basis. Two simulations were conducted for evaluation of the Alternatives:

a. January 1998 storm, starting on 13 January. This storm is considered severe with high wave energy for sediment transport. Forcing for the circulation and sediment transport models were tides, wave stress, waves, and wind. Wind fields, obtained from the NCEP, were at 0.25-deg spacing and at time interval of 6 hr. Waves were calculated by STWAVE as discussed in Chapter 5.

b. Fair-weather simulations, starting on 4 September and ending on 6 October 1998, represent typical conditions in which vessels pass through the Willapa Bay entrance. Forcing for the models were tides and representative waves and wave stress. Representative waves for this simulation are given in Table 5-7.

For comparison of velocity and sediment transport, a channel was defined for Alternative 1 based on the route that vessels took during 1998[5] and on the bathymetry that year. To facilitate comparison of physical quantities with other Alternatives, the channel was specified as generally following the 28-ft mllw contour along the vessel route. The Alternative 1 channel as defined here is shown as a dashed line in Figure 6-39. The dashed line indicates that Alternative 1 is not a design channel.

[5] Personal communication, Mr. Terry Larson, local crabber, August 1999

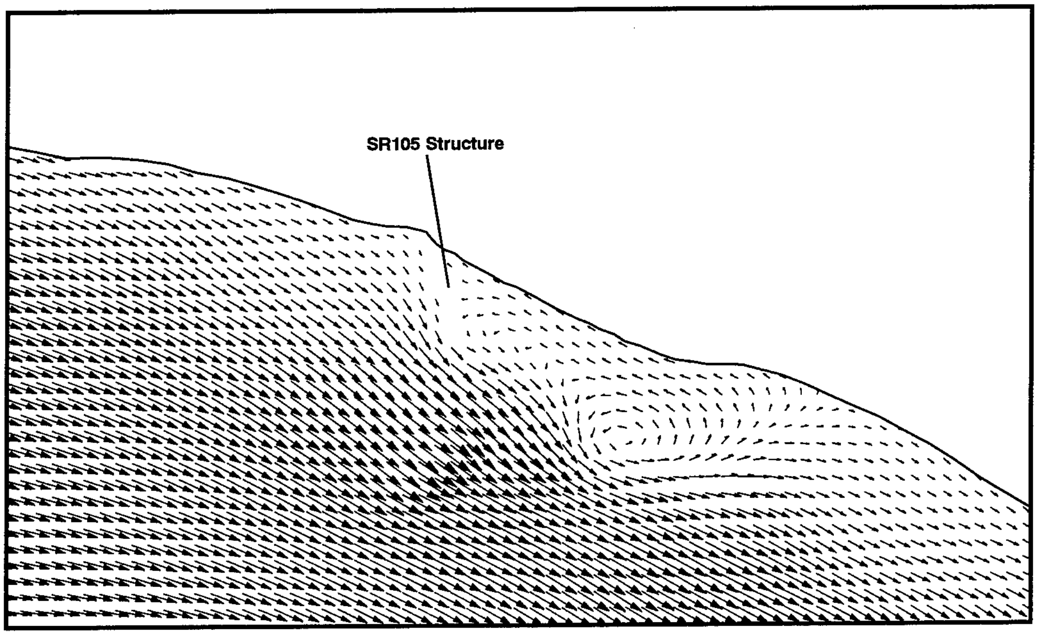

Figure 6-37. Region of SR-105 structure circulation pattern on flood tide

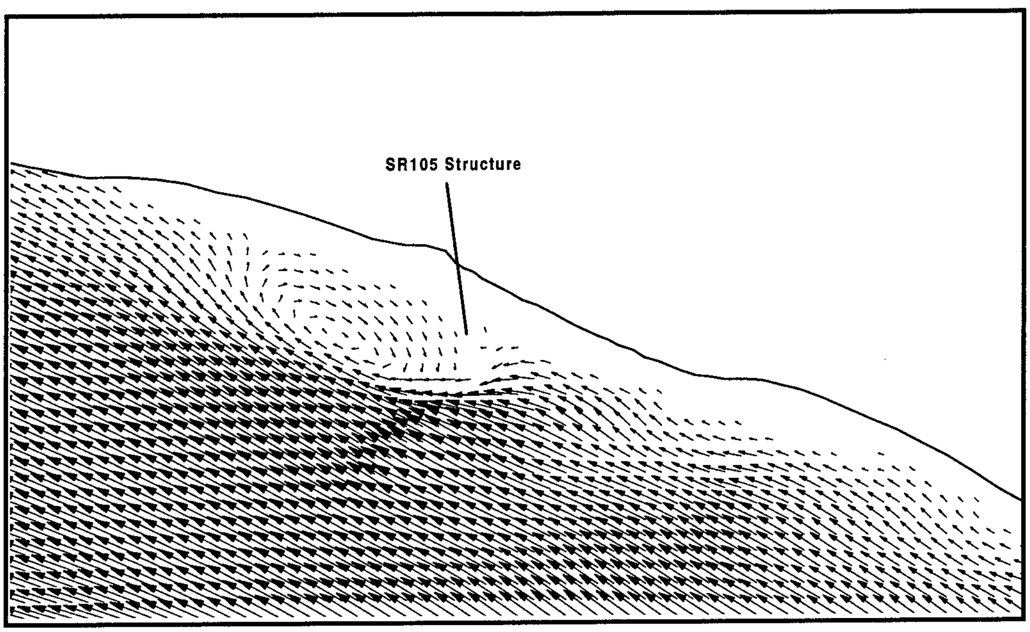

Figure 6-38. Region of SR-105 structure circulation pattern on ebb tide

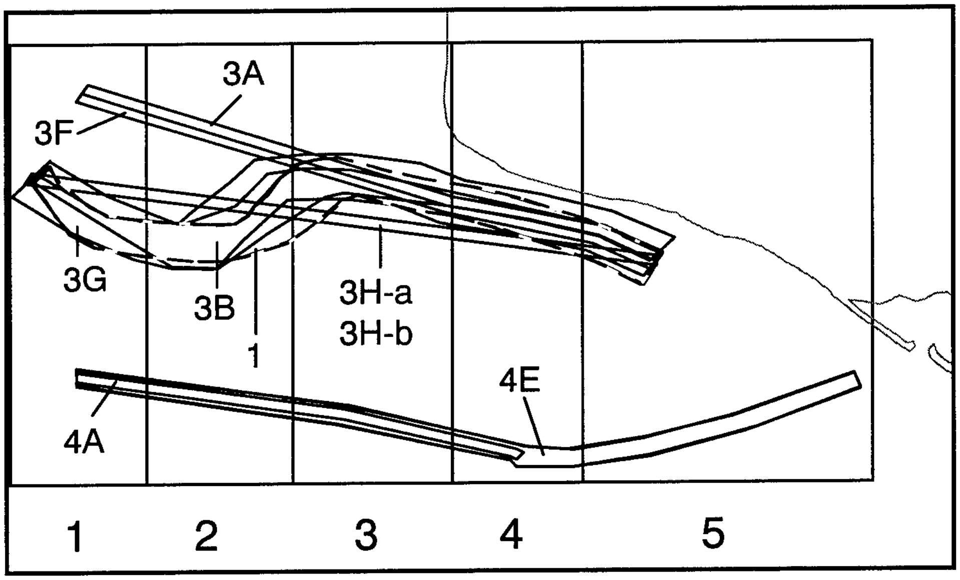

Figure 6-39. Regions of entrance defined for evaluation of current velocities in channels. Regions are defined as the east-west limits of the boxes denoted as 1, 2, 3, 4, and 5

Velocities through alternative channels

Maximum and minimum calculated speeds within the Alternative channels are given for the January 1998 storm and for September 1998 fair weather. Along- and across-channel current components are discussed. Strong along-channel currents enhance self-scouring of channels, but may pose difficulties for navigation. Strong across-channel currents can make navigation hazardous and can carry material into channels.

Locations of the reported velocities are given in five Regions shown in Figure 6-39. These Regions are as follows (reference Figure 6-14 for bottom topography):

a. Far outer entrance: outer bar, most seaward extent of channels.

b. Outer entrance: interior portion of outer bar, region of curvature of S-channel.

c. Midentrance: transition area from outer to inner bar.

d. Interior midentrance: inner bar.

e. Inner entrance: interior inner bar.

In addition to storm and fair weather conditions, current velocity was documented with respect to the direction of the current relative to the channel axis and for maximum and minimum speed. Along- and across-channel currents are specified in Tables 6-8, 6-9, and 6-10. Currents are assigned "along" or "across" designations by their dominant velocity component relative to the channel axis. Thus, the range of direction for velocity specified as "along channel" can range from parallel to the channel to an angle of 45 deg from parallel. The range of direction for "across-channel" velocity can range from

Table 6-8
Maximum and Minimum Velocity in Alternative Channels for Storm[1]

Alternative	Maximum Speed, m/sec	Direction[2] of Maximum	Region[3] of Maximum	Minimum Speed, m/sec	Direction of Minimum	Region of Minimum
1	3.7	Along	3	<0.1	Mixed	3, 4
3A	3.7	Along	3	<0.1	Mixed	4, 5
3B	3.7	Along	3	<0.1	Mixed	1
	3.1	Across	2	<0.1	Across	3, 4
3F	3.7	Along	3	<0.1	Along	2
				<0.1	Across	4, 5
3G	3.3	Along	3	<0.1	Mixed	5
	3.0	Across	2			
3H-a	3.4	Along	2	<0.1	Across	1, 2, 4
3H-b	3.4	Along	2	<0.1	Mixed	1
				<0.1	Across	3, 4
4A	3.2	Along	2	<0.1	Across	2
4E	3.2	Along	3	<0.1	Along	1, 5

[1] January 1998 storm simulation, duration = 15 days.

[2] Along or across channel at corresponding Region. Along-channel is defined as current directed at angles ranging from 0 to 45 deg relative to channel axis. Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis. Mixed indicates both along- and across-channel velocity.

[3] Regions defined in Figure 6-39.

Table 6-9
Maximum and Minimum Along-Channel[1] Velocity in Alternative Channels for Fair Weather[2] at Peak Ebb and Flood Tidal Current

	Max Ebb		Min Ebb		Max Flood		Min Flood	
Alternative	Speed, m/sec	Region	Speed, m/sec	Region	Speed, m/sec	Region	Speed, m/sec	Region
1	2.6	4	0.5	2	1.6	3, 4	0.3	2
3A	2.6	3	0.6	1	1.3	3, 4, 5	0.3	1
3B	2.6	3, 4	0.4	2	1.4	3, 4	0.2	2
3F	2.6	3	0.7	1	1.4	3	0.3	1
3G	2.4	4	0.4	2	1.3	3, 5	0.2	2
3H-a	2.0	4, 5	0.5	2	1.5	5	0.5	1, 2
3H-b	1.9	3, 4	0.6	2	1.3	3, 5	0.5	1, 2
4A	2.8	3	0.3	1	1.8	4	0.3	1
4E	1.9	4	0.3	1, 2	1.8	4	0.4	1, 2

[1] Along-channel is defined as current directed at angles ranging from 0 to 45 deg relative to channel axis.

[2] September 1998 fair weather simulation, duration = 31 days.

Table 6-10
Maximum and Minimum Across-Channel[1] Velocity in Alternative Channels for Fair Weather[2] at Peak Ebb and Flood Tidal Current

	Max Ebb		Min Ebb		Max Flood		Min Flood	
Alternative	Speed, m/sec	Region	Speed, m/sec	Region	Speed, m/sec	Region	Speed, m/sec	Region
1	1.6	2, 3	1.1	3	1.0	3	0.5	2
3A	--[3]	--	--	--	0.7	2, 3	0.7	2, 3
3B	1.6	2	0.4	2	1.0	2	0.5	2
3F	--	--	--	--	--	--	--	--
3G	1.5	2	0.4	2	1.0	2	0.2	1, 2
3H-a	0.8	1, 2	0.7	1, 2	0.8	3	0.4	2
3H-b	2.3	5	0.6	2	1.7	5	0.4	2
4A	--	--	--	--	--	--	--	--
4E	1.8	2	0.4	2	1.4	5	0.6	2

[1] Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis.

[2] September 1998 fair weather simulation, duration = 31 days.

[3] "--" indicates negligible current.

perpendicular to the channel to 45 deg from perpendicular. In many instances, currents were angled at approximately 30 deg from channel-parallel. Although these currents contain cross-channel components of flow, they were considered as predominantly along-channel currents.

Maximum and minimum velocities within Alternatives for the January 1998 storm are listed in Table 6-8. During this storm, currents at the Willapa Bay entrance were strongly modified by waves. Velocities were selected from hourly snapshots during 12 hr of peak wave conditions. Current fields during this 12-hr time interval for each Alternative are shown in Appendix G. For all Alternatives, the maximum currents occurred in Regions 2 and 3 (Figure 6-39) and direction was along channel. Alternatives 1, 3A, 3B, and 3F had the greatest along-channel current speed, peaking at 3.7 m/sec. Weakest maximum current was in Alternatives 4A and 4E, with speed of 3.2 m/sec. Alternatives 3B and 3G had persistent cross currents in Region B that reached at least 3.0 m/sec, and these are also listed in Table 6-8.

Minimum current speed for the January 1998 storm was less than 0.1 m/sec for all Alternatives. Locations of minimum currents covered all Regions. Minimum current directions did not favor a particular along- or across-channel direction overall.

Maximum and minimum calculated along- and across-channel currents for fair weather are listed in Tables 6-9 and 6-10, respectively. Current speeds are given for peak ebb and flood current in the entrance. Appendix G provides tables of maximum and minimum current speed in each Region of each Alternative for both along- and across-channel water movement. In some cases, the maximum

and minimum current for ebb or flood within an Alternative will occur in the same Region and owes to velocity variation along the channel.

Maximum along-channel ebb currents for Alternatives were typically contained in Regions 3 and 4. For Alternatives 1, 3A, 3B, 3F, and 3G, these maxima were within the North Channel. Alternative 4A was found to have the strongest calculated maximum along-channel ebb current at 2.8 m/sec, although Alternatives 1, 3A, 3B, 3F, and 3G also had strong currents at 2.6 m/sec. Weakest maximum along-channel ebb speeds were found for Alternatives 3H-b and 4E at 1.9 m/sec. Minimum along-channel speeds were located in Regions 1 and 2, on the outer bar. Alternative 3F had the strongest minimum along-channel current at 0.7 m/sec, and Alternatives 4A and 4E had the weakest at 0.3 m/sec.

Maximum along-channel currents during flood tide were weaker than those of ebb tide for all Alternatives, indicating that the channels identified for the Alternatives would be ebb dominated. The Alternative 1 channel was ebb dominated during 1998. Maximum along-channel flood speed was located in Regions 3, 4, and 5. This maximum was in the North Channel for Alternatives 1, 3A, 3B, 3F, 3G, and 3H-a. Alternatives 4A and 4E had the greatest maximum along-channel speed at 1.8 m/sec, and Alternatives 3A, 3G, and 3H-b had the weakest at 1.3 m/sec.

Minimum along-channel flood speeds were located in Regions 1 and 2 on the outer bar. Of these currents, the greatest were in Alternatives 3H-a and 3H-b at 0.5 m/sec, and the weakest were in Alternatives 3B and 3G at 0.2 m/sec.

Maximum and minimum across-channel current speeds for each Alternative are listed in Table 6-10. The presence and location of cross currents are more variable than those of along-channel currents. Across-channel currents can occur in channel locations other than those provided in Table 6-10. Appendix G contains tables listing information on across-channel currents for all Regions of each Alternative.

Maximum ebb across-channel current, where it exists, was located in Region 2 with the exception of Alternative 3H-b. Alternatives 3F and 4A did not experience crosscurrents at peak ebb and peak flood tide. Because Region 2 is in an area of breaking waves, complex current patterns are formed and cross currents can occur at channels. Alternatives 1, 3B, and 3G were calculated to have persistent and strong crosscurrents within the S-curve, which may pose difficulties for vessels traversing that area. Alternative 3H-b has strongest maximum crosscurrent in Region 5 where this channel overlaps with the North Channel. Strongest maximum across-channel current was located in Alternative 3H-b with a speed of 2.3 m/sec. This strong crosscurrent is located at the position where vessels would enter or exit this Alternative on the bay side and may present difficulties in navigating to and from it and the North Channel. Weakest ebb crosscurrent speeds were calculated for Alternative 3H-a in Regions 1 and 2 at 0.8 m/sec. Maximum ebb crosscurrents in Alternatives 1, 3B, and 3G are stronger in Region 2 than those of Alternative 3H-a indicating that Alternative 3H-a may be easier to navigate in this Region during ebb tide. Alternative 3A did not have crosscurrents during ebb tide.

Minimum ebb across-channel current occurs primarily in Region 2, with the exception being Alternative 1 in which the minimum occurs in Region 3. The

greatest minimum occurs in Alternative 1 with a speed of 1.1 m/sec, and the weakest occurs in Alternatives 3B, 3G, and 4E with speeds of 0.4 m/sec.

Maximum flood across-channel currents are distributed within Regions 2, 3, and 5. Alternatives 3B and 3G have maximum crosscurrents within the S-curve. Alternative 3H-b has maximum crosscurrents in Region 5, at its intersection with the North Channel. The strongest maximum flood across-channel current is in Alternative 3H-b with speed of 1.7 m/sec, and the weakest occurs in Alternative 3A at 0.7 m/sec.

Minimum flood across-channel currents occur in Regions 1, 2, and 3. The strongest is in Alternative 3A with a speed of 0.7 m/sec, and the weakest is in Alternative 3G with a speed of 0.2 m/sec.

The number of Regions in which crosscurrents were calculated to occur at peak ebb and flood during fair weather are listed in Table 6-11. Vessels traveling through Alternatives could encounter crosscurrents within the number of Regions listed in the table (a full listing of along- and across-channel currents for fair weather is presented in Appendix G). The type of vessel combined with the strength of the crosscurrent would determine whether or not a potential hazard might exist. For evaluation purposes, smaller numbers of Regions with cross-currents are favorable.

Table 6-11
Number of Regions in Alternative Channels with Crosscurrents[1] in Fair Weather[2] at Peak Ebb and Flood Tide[3]

Alternative	Number of Regions, Ebb	Number of Regions, Flood
1	2	2
3A	0	2
3B	2	2
3F	0	0
3G	1	3
3H-a	2	3
3H-b	3	4
4A	0	0
4E	3	2

[1] Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis.

[2] September 1998 fair weather simulation; duration = 31 days.

[3] Number of Regions computed from Tables G-1 through G-9 in Appendix G.

Alternatives 1, 3B, and 3G exhibit reversals of crosscurrent in Region 2 (Figures G-18, G-20, and G-22). The S-curve has an area in which the current flows to the northwest then rotates around forcing flow toward the southeast. The northwest and southeast crosscurrents are adjacent to one another within the Alternative channels. The oppositely-directed crosscurrents commence when the tidal current switches from ebb to flood and endures for approximately 5 hr. A vessel traversing this Region during this interval in the tidal cycle may experience crosscurrents in one direction followed by crosscurrents from the opposite direction.

To illustrate the velocity patterns within each of the design channels, vector plots are presented in Appendix G at hourly intervals over two 12-hr time intervals. The first time interval occurs during peak wave conditions in the January 1998 storm. The second time interval is during fair weather conditions from the September 1998 simulation. Both sets of plots show tide- and wave-generated currents. These plots illustrate the time-varying currents including intensification of currents by waves, formation and dissipation of eddies, and overall circulation within and near each Alternative.

Deposition and erosion

Bathymetry change was calculated by the sediment transport model for two situations: a 15-day storm simulation during January 1998 and a 31-day fair weather condition from 4 September through 6 October 1998. These two conditions were simulated to give estimates of maximum and minimum bathymetry change expected to occur over the time scale of weeks. Volume-change calculations were conducted to determine the amount of material that was deposited into the proposed Alternative channels. Estimates of shoaling volume for each Alternative were made by calculating the volume difference between the bottom elevation at the end of the simulation and the initial bottom elevation. Shoaling volumes were calculated within the Alternative channels. Table 6-12 lists the volume deposited into the channel for each Alternative and each condition.

Table 6-12
Deposition Volume Calculated for Alternative Channels

Alternative	Storm[1] Volume Deposited, yd^3	Fair Weather[2] Volume Deposited, yd^3
1	1,500,000	420,000
3A	280,000	95,000
3B	870,000	260,000
3F	600,000	190,000
3G	1,630,000	450,000
3H-a	300,000	66,000
3H-b	340,000	78,000
4A	270,000	87,000
4E	320,000	144,000

[1] January 1998 storm simulation, duration = 15 days.
[2] September 1998 fair weather simulation, duration = 31 days.

The greatest shoaling volume was calculated for Alternative 3G for both storm and fair weather conditions (1,630,000 and 450,000 yd^3, respectively). The calculated volumes deposited into Alternatives 1, and 3G channel during the January 1998 storm were almost double that of Alternative 3B, which had the third greatest shoaling volume. Smallest shoaling volumes were calculated for Alternatives 4A and 3A for the storm and for Alternatives 3H-a and 3H-b during fair weather. Volumes calculated are consistent with dredging volumes for the Pacific Ocean coast of the United States. Calculated shoaling volumes are appropriate for this engineering study, but may not be applicable for other studies.

Contour plots of depth change for Alternatives are given in Appendix G for the January 1998 storm and for fair weather. Patterns of deposition and erosion are shown and shoaling hot spots can be discerned.

Distribution and location of deposited material are central factors in maintenance dredging. Table 6-13 lists the calculated sediment shoaling (reduction in depth) in each Region of the Alternatives for the January 1998 storm. The storm was selected for comparison of shoaling between Alternatives because the high-energy waves during storms induce more material transport than during fair weather. Deposition on the outer bar in Region 1 was minimal, with Alternatives 3A and 3F having small deposits of material. In Region 2, deposition was variable between Alternatives. Alternatives 1 and 3G had significant shoaling, greater than 8 ft, which occurred within the S-curve in an area of strong crosscurrents. Alternative 3B also had significant shoaling, 5 ft, within the S-curve area. Moderate shoaling, 3 ft, was calculated for Alternatives 3H-a, 3H-b, and 4E. Alternative 4A had minor shoaling at 1 ft. Dredging in Region 2 may be difficult because of the presence of breaking waves.

Table 6-13
Shoaling in Alternative Channels for January 1998 Storm[1]

	Maximum Shoaling in Region, ft				
Alternative	**1**	**2**	**3**	**4**	**5**
1	--	≥8	≥8[2]	≥8	4
3A	1	--	4	--	≥8
3B	--	5	4	1	≥8
3F	2	--	4	--	≥8
3G	--	≥8	4	1	≥8
3H-a	--	3	≥8	2	4
3H-b	--	3	≥8	2	5
4A	--	1	≥8	--	--
4E	--	3	3	1	--

[1] January 1998 storm simulation, duration = 15 days.
[2] Deposition in localized mounds.

In Region 3, sediment shoaling varied between a large amount in Alternatives 1, 3H-a, 3H-b, and 4A at more than 8 ft, and moderate in the remaining Alternatives (3 to 4 ft). Within Alternative 1, shoaling in Region 3 was limited to relatively small mounds that did not extend across the channel (see Figure G-27). Dredging in Region 3 may be difficult because of the frequent presence of breaking waves.

In Region 4, there was little to no sediment shoaling in all Alternative channels with the exception of Alternative 1. Shoaling of more than 8 ft was calculated in Region 5 for Alternatives 3A, 3B, 3F, and 3G, and moderate shoaling (4 to 5 ft) was calculated for Alternatives 1, 3H-a and 3H-b. The area of shoaling was common to Alternatives within Region 5 and was located in the North Channel seaward of the dike-and-groin structure. The presence of the structure may have contributed to calculation of deposition near it. Alternatives 4A and 4E had no shoaling in Region 5 (Alternative 4A does not extend into Region 5).

Elevation change greater than 8 ft was calculated to occur in Regions 1 through 4 for Alternatives 1, 3G, 3H-a, 3H-b, and 4A during the storm. Deposition in Alternative 4A did not extend fully across the channel, as it did for the four other Alternatives. The deposition calculated for Alternatives 3A, 3B, 3F, and 3G in Region 5 does not present a navigation hazard because the North Channel is deep and self-maintaining in this area.

Salinity Change in Willapa Bay

Modeling of salinity was conducted to evaluate changes in salinity within Willapa Bay that would arise from construction of a navigation channel. Comparisons of salinity at points in Willapa Bay were made between the existing condition and each Alternative from a 5-day simulation of salinity transport. Tides were the only forcing applied in the salinity simulations. Salinity modeling was conducted by running the existing condition for 14 days so that the model was equilibrated from the initial condition. The salinity distribution at the 14th day served as the initial condition for the 5-day simulations. Simulations were conducted from 15 through 20 August 1997, during spring tide. Salinity boundary conditions at the Willapa River and Nahcotta River were set to 22 ppt and 27 ppt. These salinity values are representative of monthly mean salinity values for the month of August 1997, when the bay was vertically well mixed (Newton et al. 1998). Selection of a time when the bay was vertically well mixed is an appropriate condition for simulating salinity transport with depth-integrated hydrodynamic and transport models.

Because salinity is a significant factor for oysters, stations for analysis of salinity change were selected at tidal flats at 11 locations in Willapa Bay. Salinity change was also calculated at eight stations in Willapa Bay channels. Figure 6-40 shows the 19 stations where salinity change was calculated. Stations were widely distributed over the bay to identify any area where significant salinity change would occur.

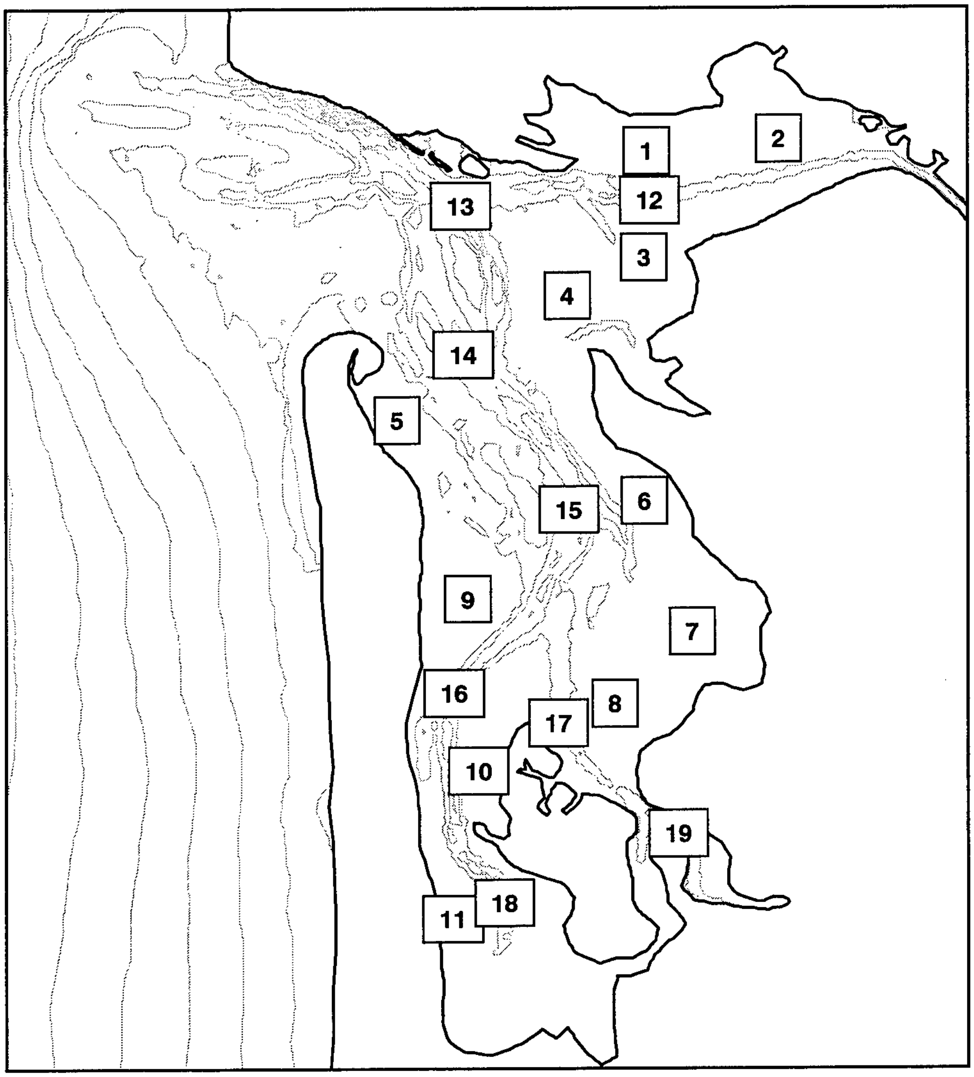

Figure 6-40. Stations for salinity change analysis

Salinity change was determined by calculating the difference appearing between the individual Alternatives and the existing condition over the 5-day simulation. Positive values of salinity difference indicate that salinity is greater for the Alternative than for the existing condition. At all Alternatives, the mean salinity change over the 5-day simulation interval was calculated to be <0.1 ppt. Greatest salinity change was calculated for Alternatives 3H-a and 3H-b in which the average salinity was decreased by 0.2 ppt at Station 3. The remaining Stations had calculated mean salinity changes of <0.1 ppt. These small differences are well within variability of the system and much less than the variation in vertical profile of salinity as measured, for example, in this study (Chapter 4).

Summary of Circulation and Transport Modeling

Circulation and transport modeling were conducted with the goal of evaluating Alternatives for a safe and reliable entrance channel at Willapa Bay. Currents and water level were calculated with the finite-element model ADCIRC. The model grid was developed from 1998 bathymetry at Willapa Bay, and high resolution was specified in the entrance. Resolution for each Alternative was specified so that calculation points were placed every 250 ft within the channels.

Wave-induced currents were calculated by including wave stresses, computed from STWAVE-calculated wave parameters, in the momentum equations within ADCIRC. Wave-induced currents caused a substantial change in the circulation patterns compared with tidal flow only. In particular, wave-induced currents typically cause cross-channel flows and sediment deposition in channels. Sediment transport was implemented in the ADCIRC model using the Ackers and White total-load formulation. Both wave-induced currents and sediment transport calculation were verified for the situation of a planar beach.

Modeling was conducted to calculate change in salinity within Willapa Bay for each Alternative. Tides were the only forcing applied in the salinity simulations. Calculated salinity for each Alternative showed no significant change from the existing condition. The greatest mean salinity change, -0.2 ppt occurred at one tidal flat location on the Willapa River for Alternatives 3H-a and 3H-b.

To evaluate proposed channel Alternatives for Willapa Bay, factors related to navigation and maintenance dredging were calculated. With respect to circulation, the most significant factor for navigation is the presence of cross-currents. For maintenance dredging, the most significant factor is volume of material deposited into the channel. As a summary of work presented in this chapter, Table 6-14 lists Alternatives based on calculated crosscurrents during fair weather and deposition during the January 1998 storm. Alternatives are grouped into categories for maximum crosscurrent speed and shoaling volumes. Alternatives within these groups are listed in order of increasing speed or volume, as calculated by the model.

Alternatives were also listed with regard to crosscurrent speed and number of Regions in which crosscurrents could be encountered. Alternatives 3A, 3F, 4A, and 3H-a had weakest crosscurrent speeds, whereas Alternatives 3A, 3F, and 4A had the fewest number of Regions in which crosscurrents might be experienced by vessels. Thus, with respect to navigation, Alternatives 3A, 3F, and 4A would be the preferable Alternatives because of least influence of crosscurrents on vessels.

Table 6-14 lists three groups of Alternative channels by calculated volume of material deposited into them during the January 1998 storm. The storm was selected for this comparison because high waves during storms initiate suspension of material into the water column and increase the current velocity for material transport. Alternatives with ranks of 1 are 4A, 3A, 3H-a, 4E, and 3H-b, as the least amount of material was deposited into their channels during the storm.

Table 6-14
Grouping of Alternatives Based on Navigation and Shoaling Factors

Navigation: Crosscurrents in Alternative channels during fair weather			
Max Speed during Ebb , m/sec	**Alternatives Ebb Tide**	**Max Speed during Flood, m/sec**	**Alternatives Flood Tide**
0.0 – <1.0	3A, 3F, 4A, 3H-a	0.0 – <1.0	3F, 4A, 3A, 3H-a
1.0 - <2.0	3G, 1, 3B, 4E	1.0 - < 2.0	1, 3G, 3B, 4E, 3H-b
≥2.0	3H-b	--	--
Number of Regions with Cross currents on Ebb Tide	**Alternatives Ebb Tide**	**Number of Regions with Crosscurrents on Flood Tide**	**Alternatives Flood Tide**
0	3A, 3F, 4A	0	3F, 4A
1	3G	1	--
2	1, 3B, 3H-a	2	1, 3A, 3B, 4E
3	3H-b, 4E	3	3G, 3H-a
4	--	4	3H-b

Shoaling volume for January 1998 storm	
Volume, cu yd	**Alternative**
270,000 – 340,000	4A, 3A, 3H-a, 4E, 3H-b
600,000 – 870,000	3F, 3B
1,500,000 – 1,630,000	1, 3G

The four most favorable Alternatives are Alternatives 3A, 3F, 3H-a, and 4A, if both navigation and shoaling considerations are combined. Although cross-currents in Alternative 3F were negligible, the volume of material deposited into it during the storm was more than twice that of the other three most favorable Alternatives. Thus, Alternative 3F can be eliminated from evaluation based on the present analysis.

The three most favorable Alternatives determined from this analysis are 3A, 4A, and 3H-a. Table 6-15 summarizes the properties of these three Alternatives.

Table 6-15
Properties of Most Favorable Alternative Channels Based on Crosscurrent and Shoaling Calculations

Alternative	Crosscurrents[1]	Shoaling[2, 3]
3A	3 Regions on flood tide	No Regions of great shoaling
4A	None	Great shoaling in Region 3
3H-a	3 Regions on ebb tide 4 Regions on flood tide	Great shoaling in Region 3

[1] All three Alternatives had crosscurrents of 1.0 m/s or less.

[2] Shoaling in Region 5 in North Channel is not considered as a maintenance dredging problem.

[3] Great shoaling indicated calculated depth reduction of at least 8 ft within the Alternative channel during the January 1998 storm.

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7 Evaluation of Navigation Channel Alternatives[1]

The objective of this study is to determine the engineering feasibility of maintaining a reliable navigation channel through the entrance to Willapa Bay. This chapter discusses the factors or criteria relevant for judging navigation reliability. In addition to navigation safety, selection of feasible alternatives also involves the cost of dredging, both through possible beneficial uses of the dredged material and through the stability of the adjacent beach as part of the overall inlet system.

The preceding chapters and associated appendices describe technical details and results of the individual study tasks that generated information for evaluating alternative channel designs. This chapter summarizes those results and adds integrating interpretations. Material is presented in tabular and graphical form concerning the following general categories:

a. Project operation and maintenance.

b. Navigation safety.

c. Environment and beneficial uses.

Project operation and maintenance is listed first because if the project is too costly, it cannot go to construction, so that navigability is not an issue. Such reasoning was applied in the screening process that reduced the original 19 alternatives to a smaller number for detailed evaluation.

Alternatives

The alternatives are described in Chapter 2. The screening process presented in Chapters 5 and 6 and elsewhere in the report produced three basic alternative groups, as shown in Figure 7-1. Depths are referenced to mean lower low water (mllw).

(1) North Fairway:

Alternative 3A: 28 ft-deep by 500 ft-wide channel, fixed location.

[1] Written by Mr. William C. Seabergh and Dr. Nicholas C. Kraus, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS, and by Mr. Hiram T. Arden, U.S. Army Engineer District, Seattle, Seattle, WA.

Alternative 3B: 28-ft-deep by 500 ft-wide migrating channel, with minimum 1,500-ft width in S-curve.

Alternative 3F: 38-ft deep by 1,000-ft wide channel, fixed location.

Alternative 3G: 38-ft deep by 1,000 ft wide migrating channel, with minimum 1,500-ft width in S-curve.

(2) State Route (SR)-105 alternatives:

Alternative 3H-a: 28-ft deep by 500 ft-wide channel.

Alternative 3H-b: 28 ft-deep by 500 ft-wide channel, with the SR-105 dike raised from 18 ft mllw depth to 2 ft mllw depth.

(3) Middle Fairway:

Alternative 4A: 28 ft-deep by 500 ft-wide channel.

Alternative 4E: 38 ft-deep by 1,000 ft-wide channel.

Alternatives 3H-a and 3H-b were included for evaluation after new bathymetric measurements made during the course of this study indicated some deepening of the south side of the north channel just seaward of the SR 105 project. These two alternatives were added to determine if any advantages could be found in placing a navigation channel in this vicinity. Figure 7-1 shows the alternatives.

Criteria for Evaluation of Navigation Channel Alternatives

The criteria for evaluating the various channel alternatives were separated into three groups, as listed in Tables 7-1, 7-2, and 7-3. Table 7-1, dealing with operation and maintenance, comprises factors indicating estimated and calculated sediment volumes requiring removal from the navigation channel. In Willapa Bay, the high-energy waves, large tidal range, and strong tidal current, coupled with environmental considerations, combine to give relatively small windows of opportunity for dredging during the year. Evaluation of alternatives must consider the amount of maintenance dredging required both for the cost involved and the limited time dredging can be performed.

Two approaches were taken for evaluating the magnitude of sediment that would need to be dredged to maintain a safe and navigable channel for the subject alternative. A geomorphic approach examined long-term or life-cycle project dredging requirements. This life-cycle dredging was based on a 50-year project lifetime, and estimates were developed from results of Chapter 3. The second factor for the operation and maintenance evaluation came from the results of the numerical modeling discussed in Chapter 6. Volumes deposited in the channel during a representative severe winter storm and volumes deposited during a 31-day period of "typical" oceanic conditions defined the conditions for this evaluation.

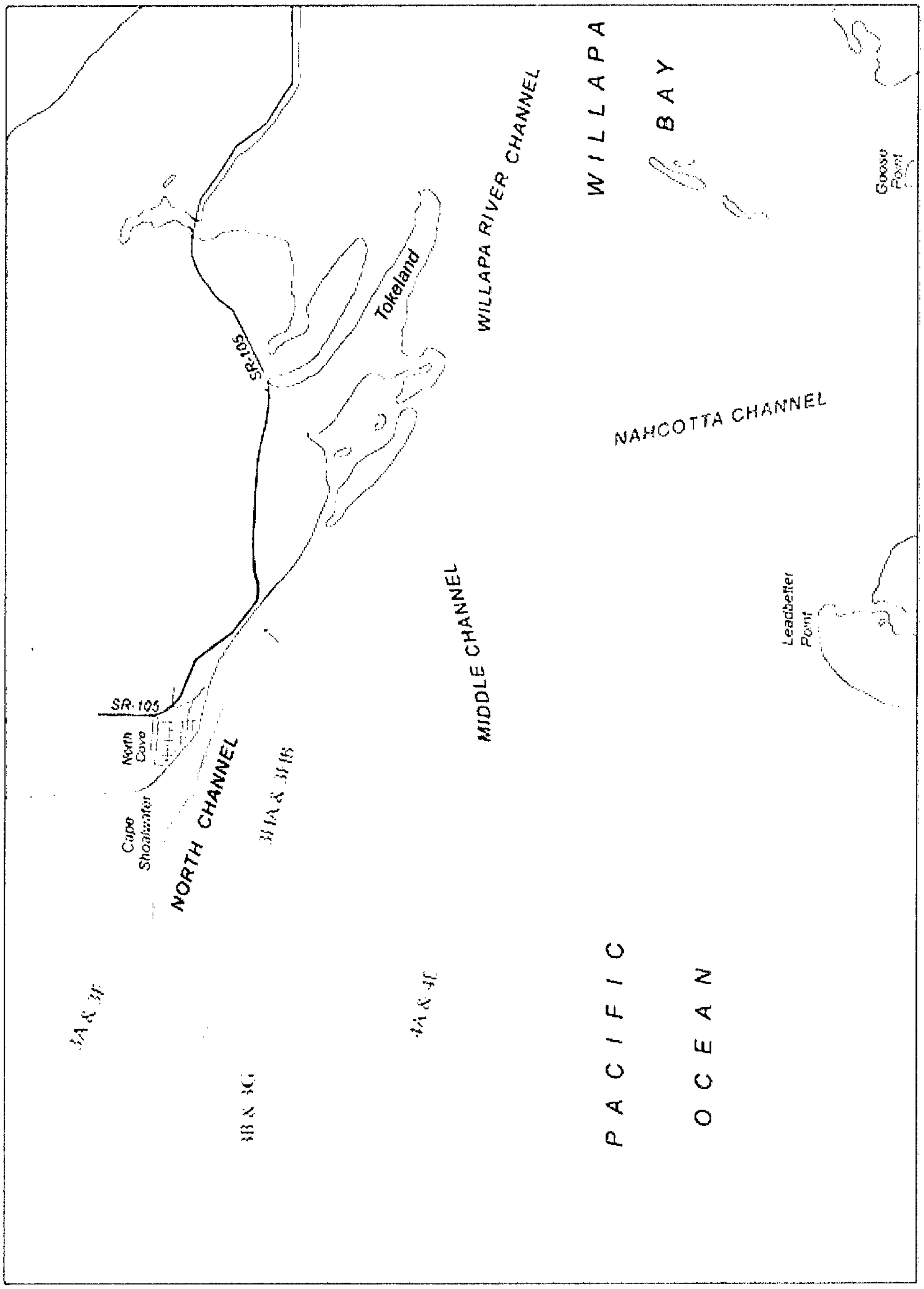


Figure 7-1. Alternatives for evaluation

Another factor of operation and maintenance relates to the estimated cost for surveying the inlet channel. The U.S. Army Corps of Engineers must provide guidance to the U.S. Coast Guard (USCG) for marking channels to identify safe access through the inlet. Before moving any channel markers, the USCG must receive a channel survey. An Alternative which has a less stable location or requires surveying under relatively more severe wave conditions will cost more to survey. The variations in these costs were not significant enough with respect to the costs associated with dredging, and were not included in Table 7-1.

Table 7-2 lists navigation safety factors. The first criterion is derived from results of the modeling of surface waves (Chapter 5) and the discussion of wave direction with respect to channel orientation in Chapter 2. Wave heights averaged along the channel are included in the table for 9-ft incident ocean waves. This wave height was considered a typical operational maximum for tug-and-barge traffic. Wave angles greater than 45 deg with respect to the channel axis are considered problematic for vessels. The Certainty of Depth factor evolved from the historic analysis of channel location discussed in Chapter 3 and channel capacity to maintain a 28-ft depth. Information contributing to the Channel Curvature and Variation in Location factor was developed from the historical analysis of Chapter 3. Information for the Alignment of Current With Channel factor was determined by examining visual results of numerical simulations of combined tidal circulation and surface waves model (Chapter 6).

Information contained in Table 7-3 concerns beneficial-use factors of dredged material and erosion potential at North Cove. Potential dredged-material placement sites are discussed in Appendix H. These results led to an overall examination of the sites for each alternative, based on availability, cost, capacity, and beneficial use. Numerical modeling of tidal currents provided information to compare with existing conditions for North Cove Erosion Potential.

The Evaluation

Table 7-1 summarizes the evaluation of the operation and maintenance factors considered. The "1998 Initial Dredging Volume" is the first item following the "Description" column. Presently, the S-curve channel of Alternative 3B requires minimal dredging, because it is the existing configuration. It is now in a filling mode, however, because as the "Annual Net Shoaling in Location of Potential Channel" column indicates, 1.0 million cu yd of fill occurred between 1998 and 1999. Alternative 3A requires 800,000 cu yd of dredging and is in a scour mode, as this potential location had 1.1 million cu yd of scour in the 1998-1999 time period. The deeper versions of Alternatives 3A and 3B, 3F and 3G, because of the greater initial depth, require 5.7 and 4.2 million cu yd of dredging, respectively. Alternatives 3H-a and 3H-b both require 2.1 million cu yd of dredging. The middle channel locations of Alternatives 4A and 4B need 2.2 and 12.0 million cu yd of initial dredging, respectively.

Following the aforementioned columns is the "50-Year Lifetime Maintenance Dredging Estimate," showing the projected amounts of dredging over a 50-year project lifetime, based on the geomorphic analysis of Chapter 3. The volumes are presented as an estimated range of values which could occur for a specific Alternative. The migrating channel Alternative 3B has the smallest

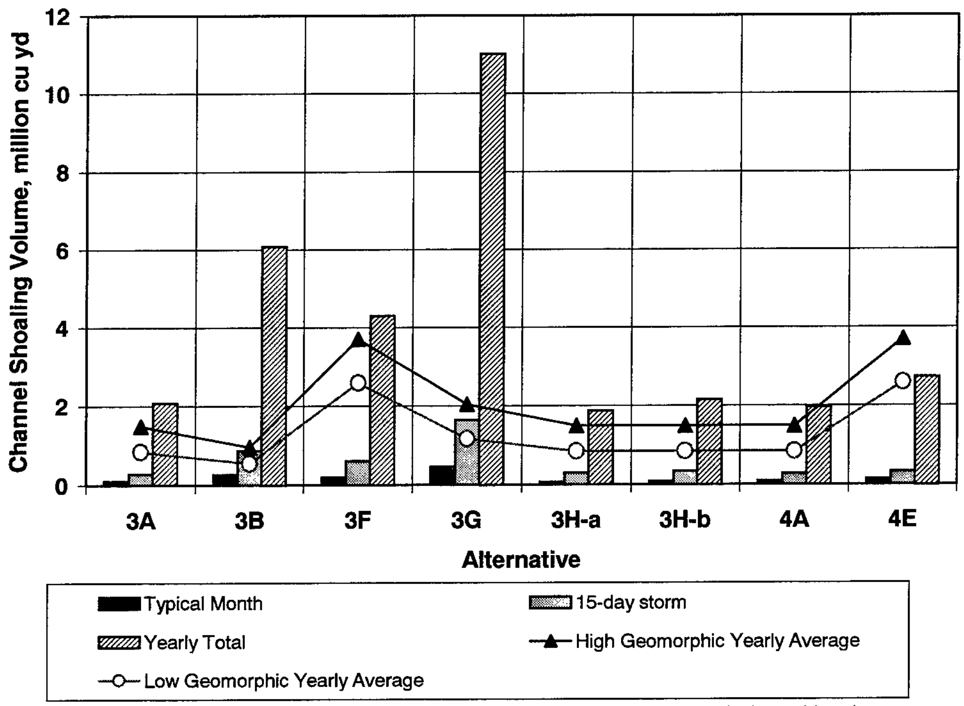
amount, a 27-48 million cu yd range over 50 years, but would probably have the greatest likelihood of providing unsatisfactory navigation conditions, as is discussed below. Alternative 3G, also a migrating channel that is deeper and wider than 3B, has an estimated range of 58-102 million cu yd over 50 years. Alternatives 3A, 3H-a, 3H-b, and 4A all have lifetime estimated shoaling volume range of 42-74 million cu yd. The deeper and wider channel of Alternative 3F has a lifetime shoaling estimate in the range of 114-185 million cu yd. These shoaling volumes are converted to annual rates and compared to numerical model results in the next paragraph.

The numerical model results of channel shoaling in Chapter 6 are listed in the sixth and seventh columns of Table 7-1. The typical condition, 31-day simulations of channel shoaling, and the 15-day storm results are shown. Alternatives 3H-a, 3A, 4A, and 3H-b were reasonably close to one another in channel shoaling. Alternative 4E had slightly greater shoaling than the previously mentioned Alternatives, and Alternatives 3F, 3B, and 3G had progressively increasing amounts of channel shoaling. To compare the geomorphic and numerical modeling results, a yearly estimate of shoaling was determined from the geomorphic 50-year estimates and, using the following procedure, the numerical results provided an annual channel shoaling estimate: the 15-day storm period was multiplied by a factor of 4 to represent a 60-day period, and the 31-day typical values were multiplied by 10 to represent a 310-day period. Adding the two shoaling results produced a yearly estimate plotted in Figure 7-2. Shoaling rates of the two approaches were in reasonable agreement except for the migrating channel Alternatives 3B and 3G, where the geomorphic approach allows channel movement until channel curvature approaches 1998 conditions. Therefore, dredging is minimal until strong channel curvature is present and the channel must be realigned.

Capacity of a channel is another central aspect in the evaluation of channel shoaling. In the past, typical dredging did not produce a channel that would maintain depth or alignment for the time span between dredging. This is the reason the wider and deeper plans of Alternatives 3F and 3G were included in the evaluation. The wider footprints of these channels capture more sediment (shoaling), but the increased capacity will produce longer time periods of navigable depth and alignment.

Table 7-2 presents navigability safety factors. The entries in this table derive from the wave tests of Chapter 5 (third and fourth columns, "Wave Height and Angle"), the historic analysis of Chapter 3 (fifth and sixth columns, "Likelihood of Maintaining Depth Over Bar," and "Channel Curvature and Variation in Location," respectively), and the tidal and wave-generated current modeling of Chapter 6 ("Alignment of Current With Channel"). In the wave height and angle columns, the large wave angles with respect to channel orientation of Alternatives 3B and 3G are highly undesirable. Alternatives 3H-a and 3H-b had small wave angles but slightly greater wave heights than Alternatives 3A, 3F, 4A, and 4E.

As shown in the "Likelihood in Maintaining Depth Over Bar" column of Table 7-2, the historic analysis favors deeper dredged and migrating northern channel Alternatives 3B, 3F, 3G, and 3H, for certainty of depth, and the middle



Note 1: Yearly total calculated as 4 times the storm volume plus 10 times the typical monthly volume
Note 2: Geomorphic volumes for Alternatives 3B and 3G assumes the channel moves and dredging only occurs during periodic readjustment of channel to north. The channel is relocated when S-curve becomes difficult for navigation.

Figure 7-2. Yearly shoaling estimates based on geomorphic analysis and numerical modeling

channels less favorably. As far as "Channel Curvature and Variation in Location" ratings, Alternative 4E is highest because of its large width and depth, and its distance from the migrating northern spit. Alternatives 3A, 3F, 3H, and 4A are considered to have reasonable success at maintaining alignment and location through dredging activity.

The fourth factor of "Alignment of Current With Channel," evaluated in Table 7-2, was determined from results in Chapter 6. Good alignment of the currents with the channel is critical for tugs towing non-powered barges. Alternatives 3A, 3F, and 4A have good flow alignment with the channel. The other alternatives have reasonable alignment except for Alternative 3B, which has the strongest crosscurrents.

Table 7-3 compares alternatives for "Beneficial Use of Dredged Material and Erosion Potential." The evaluation of dredged-material disposal sites presented in Appendix H and summarized in Chapter 2 were consulted for this table. The straight northerly channel routes of Alternatives 3A, 3F, 3H-a, and 3H-b tend to rate higher overall with regard to proximity to available disposal sites and beneficial usage for the dredged material. Alternatives 3B and 3G are at a slightly greater distance from the optimal disposal sites. Alternatives 4A and 4E require sidecast dredging or use of the Goose Point disposal site.

Table 7-1
Design Alternative Evaluation, Willapa Bay Navigation Channel Reliability Study[1], Operation and Maintenance Factors

Alternative	Description	Estimated Dredging and Maintenance – Life cycle (Geomorpologically Based), million cu yd			Sediment Transport into the Channel-Event/Short Term (Numerical Model), cu yd		Comments
		1998 Initial Dredging Volume	Annual Net Shoaling in Location of Potential Channel, 1998-1999	50-Year Lifetime Maintenance Dredging Estimate	Typical Conditions for 31 Days	Jan 98 Storm for 15-day Storm	
1	1998 existing condition						Dredging not being performed at this time. Recent (1957-1974) yearly historical amounts were as high as 610,000 cu yd, yet not maintaining a safe navigable channel for a significant time period over the dredging cycle.
3A	Dredge primarily on entrance bar, straight out and fixed in position	0.8	-1.1 (scour)	42-74	95,000	280,000	Moderate initial cost relative to others. Sediment transport into channel next to lowest.
3F	Same as 3A, except dredge to total depth of 38 ft and with width 1,000 ft	5.7	-3.8 (scour)	114-185	190,000	600,000	Lifetime dredging based on complete sediment filling and does not consider possible improved flushing of sediment from channel. Moderate sediment influx and large footprint.
3B (Migrating channel)	Modified S-curve (moderate curve)	0.04	1.0 (fill)	27-48	260,000	870,000	Significant amounts of sediment transport into channel, but channel has large footprint to absorb influx and channel allowed to migrate.
3G (Migrating channel)	Same as 3B, except dredge to total depth of 38 ft and with width 1,000 ft	4.2	2.4 (fill)	58-102	450,000	1,630,000	Significant amounts of sediment transport into channel, but channel has large footprint to absorb influx and channel allowed to migrate.
3H-a	Existing SR-105 dike	2.1	-0.8 (scour)	42-74	66,000	300,000	Minimal sediment influx. Moderate startup cost.
3H-b	SR-105 dike raised to -2 ft (3H-b)	2.1	-0.8 (scour)	42-74	78,000	340,000	Slightly higher sedimentation than 3H-a. Construction cost for raising dike. Moderate initial dredging cost.
4A	Dredge primarily on entrance bar	2.2	0.1 (fill)	42-74	87,000	270,000	Minimum channel sedimentation. Low storage capacity of channel. Moderate initial dredging.
4E	Same as 4A, except dredge to total depth of 38 ft and with width 1,000 ft	12.0	1.2 (fill)	114-185	144,000	320,000	Lifetime dredging based on complete sediment filling and does not consider possible improved flushing of sediment from channel. Large footprint and storage capabilities. High startup cost.

[1]Note: All channel depths are 26 ft mllw + 2 ft over-dredging (500 ft width at bottom) unless otherwise specified.

Table 7-2
Design Alternative Evaluation, Willapa Bay Navigation Channel Reliability Study[1], Navigability Safety Factors

Alternative	Description	Wave Height and angle		Likelihood of Maintaining Depth Over Bar	Channel Curvature and Variation in Location	Alignment of Current with Channel	Comments
		Average Along-Channel Significant Wave Height	Average Wave Angle at Worst Location	Certainty of Maintaining Depth Over Bar Between Dredging Cycles	Long-term Probability of Maintaining a Straight Channel	Do the Currents Safely Follow the Channel or are there Significant Cross-Currents	
1	1998 existing condition	1.3 m	30 deg	Low certainty	Very low	Fair alignment	Typical "S-curve" condition very difficult for safe navigation.
3A	Dredge primarily on entrance bar, straight out and fixed in position	1.4 m	23 deg	Moderate certainty	Good	Very favorable alignment	Overall good navigability.
3F	Same as 3A, except dredge to total depth of 38 ft and with width 1,000 ft	1.4 m	23 deg	High certainty	High	Very favorable alignment	Best Alternative with respect to navigability.
3B (Migrating channel)	Modified S-curve (moderate curve)	1.7 m	51 deg	High certainty	Very low	Poor alignment	Wave angle with vessel greater than 45 deg. Channel is not straight but 1,500-ft width in s-turn provides increased safety.
3G (Migrating channel)	Same as 3B, except dredge to total depth of 38 ft and with width 1,000 ft	1.7 m	51 deg	Highest certainty	Low	Poor alignment	Wave angle with vessel highly undesirable.
3H-a	Existing SR-105 dike	1.8 m	14 deg	Moderate certainty	High probability	Favorable alignment	Good alignment with respect to wave angle. Would anticipate improved alignment with current.
3H-b	SR-105 dike raised to elevation -2 ft (3H-b)	1.8 m	14 deg	High certainty	Good probability	Fair alignment	Good alignment with respect to wave angle. Would anticipate improved alignment with current.
4A	Dredge primarily on entrance bar	1.4 m	26 deg	Low certainty	Good probability	Very favorable alignment	Historical records do not support good depths in Middle Channel.
4E	Same as 4A, except dredge to total depth of 38 ft and with width 1,000 ft	1.4 m	26 deg	Moderate certainty	Very high	Fair alignment	Historical records do not support adequate depths in Middle Channel.

[1] Note: All channel depths are 26 ft mllw + 2 ft over-dredging (500 ft width at bottom) unless otherwise specified.

Table 7-3
Design Alternative Evaluation, Willapa Bay Navigation Channel Reliability Study[1], Beneficial Use of Dredged Material and Erosion Potential

		Dredging Disposal Sites	North Cove Erosion Potential	
Alternative	**Description**	**Availability, Cost, Capacity and Beneficial Uses**	**Erosion Potential by Tidal Current Erosion**	**Comments**
1	1998 existing condition	Good availability of sites, but no dredging being performed, so no beneficial uses.		Due to minimal dredging, good availability of disposal sites
3A	Dredge primarily on entrance bar, straight out and fixed in position	Reasonable availability and capacities available, best beneficial uses.	Similar to existing	Near historically approved disposal sites.
3F	Same as 3A, except dredge to total depth of 38 ft and with width 1,000 ft	Reasonable availability and capacities available, best beneficial uses.	Similar to existing	Large amount to be disposed.
3B (Migrating channel)	Modified S-curve (moderate curve)	Reasonable availability and capacities available, good beneficial uses.	Similar to existing	Near historically approved disposal sites.
3G (Migrating channel)	Same as 3B, except dredge to total depth of 38 ft and with width 1,000 ft	Reasonable availability and capacities available, good beneficial uses.	Similar to existing	Large amount to be disposed.
3H-a	Existing SR-105 dike	Reasonable availability and capacities available, best beneficial uses.	Slight reduction from existing	Near historically approved disposal sites.
3H-b	SR-105 dike raised to elevation -2 ft (3H-b)	Reasonable availability and capacities available, best beneficial uses.	Significant reduction from existing	Near historically approved disposal sites.
4A	Dredge primarily on entrance bar	Marginal availability and capacities available, good beneficial uses.	Slight reduction from existing	Side-casting necessary.
4E	Same as 4A, except dredge to total depth of 38 ft and with width 1,000 ft	Marginal availability and capacities available, good beneficial uses.	Slight reduction from existing	Side-casting necessary.

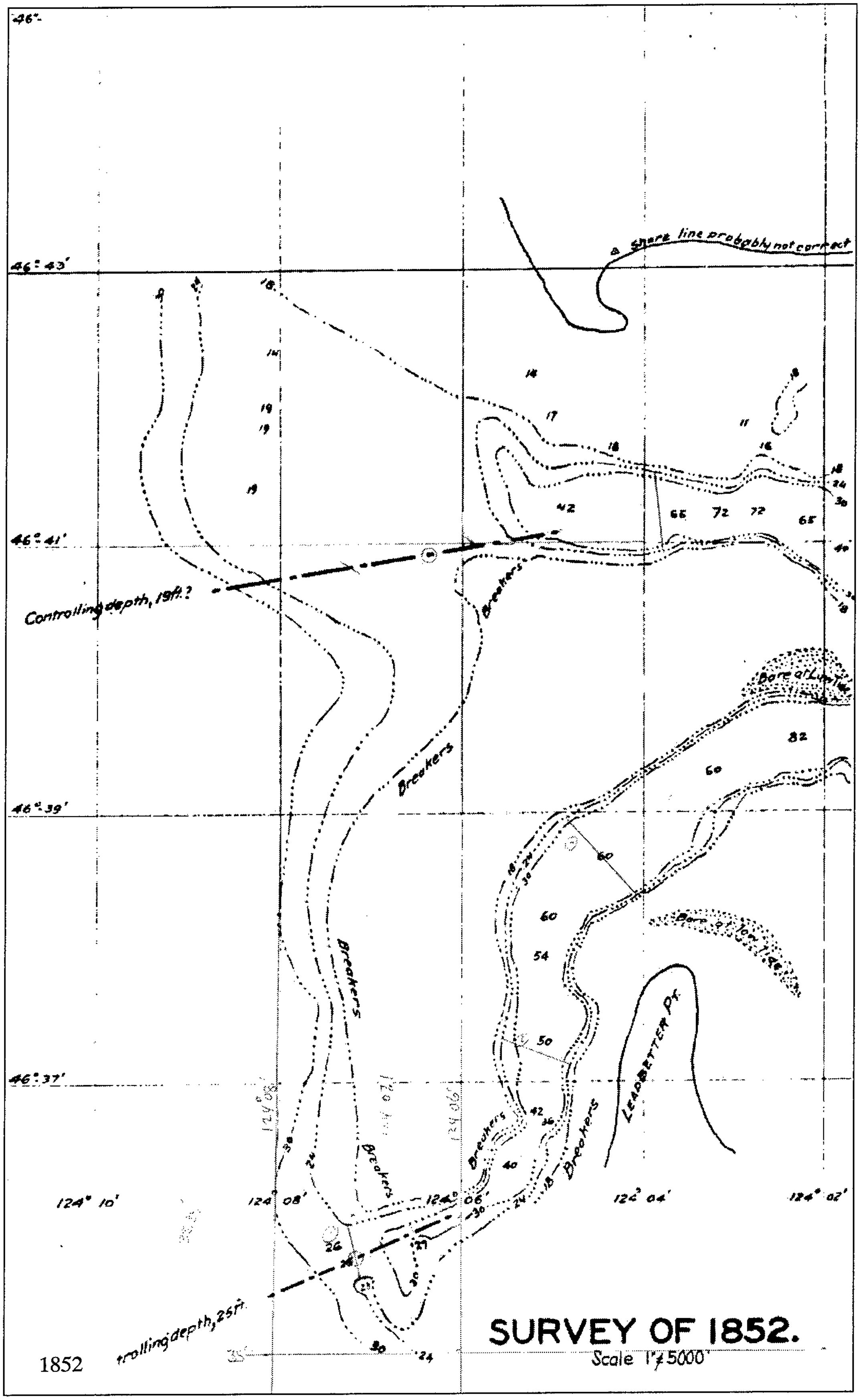
[1] Note: All channel depths are 26 ft mllw + 2 ft over-dredging (500 ft width at bottom) unless otherwise specified. Elevation referenced to mllw.

Appendix A
Historical Charts[1]

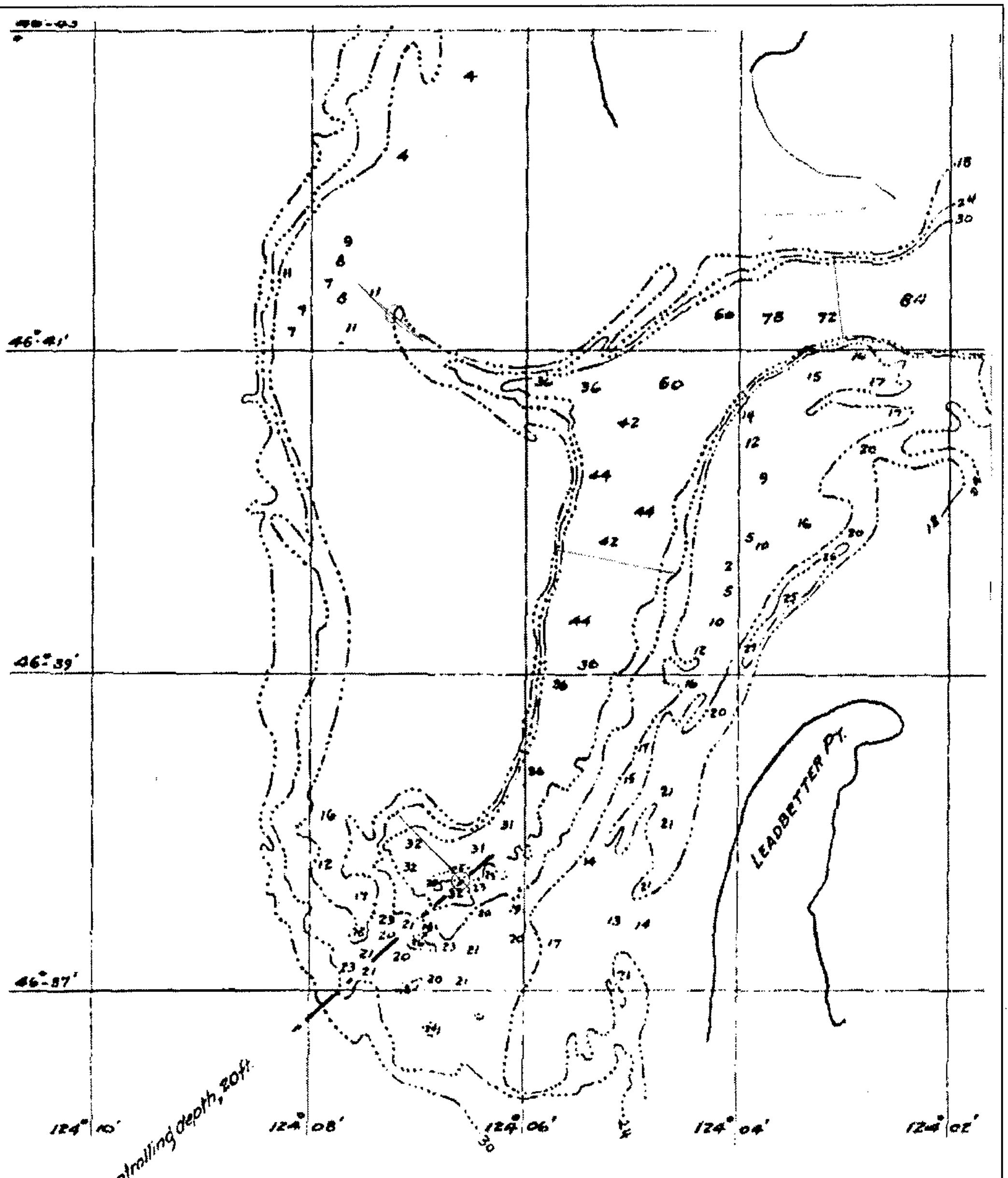
This appendix contains reduced-size copies of the 1-in.- to 2,000-ft- and 1-in.- to 5,000-ft-scale drawings compiled and analyzed in Chapter 3 of the main text.

The original drawings are stored in the archives of the U.S. Army Engineer District, Seattle. Digital data from the Seattle District surveys conducted in the 1990s were plotted in a similar style to complete the presentation here of one chart for each of the years analyzed in this study. Inquiries concerning archive holdings can be addressed to Ms. Joyce E. Rolstad, Engineering Records and Information Section, CENWS-ED-TB-RI, U.S. Army Engineer District, Seattle. P.O. Box 3755, Seattle, WA 98124-3755, (206) 764-6704.

[1] Organized by Mr. Edward B. Hands, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS. Messrs. Fulton C. Carson and Michael Martinez scanned full-size copies of the archived D- and C-size charts at 150 dots per inch for use by Mses. Mary C. Allison and Leonette J. Thomas in preparation of this appendix. Selected charts were digitized for volume calculations.



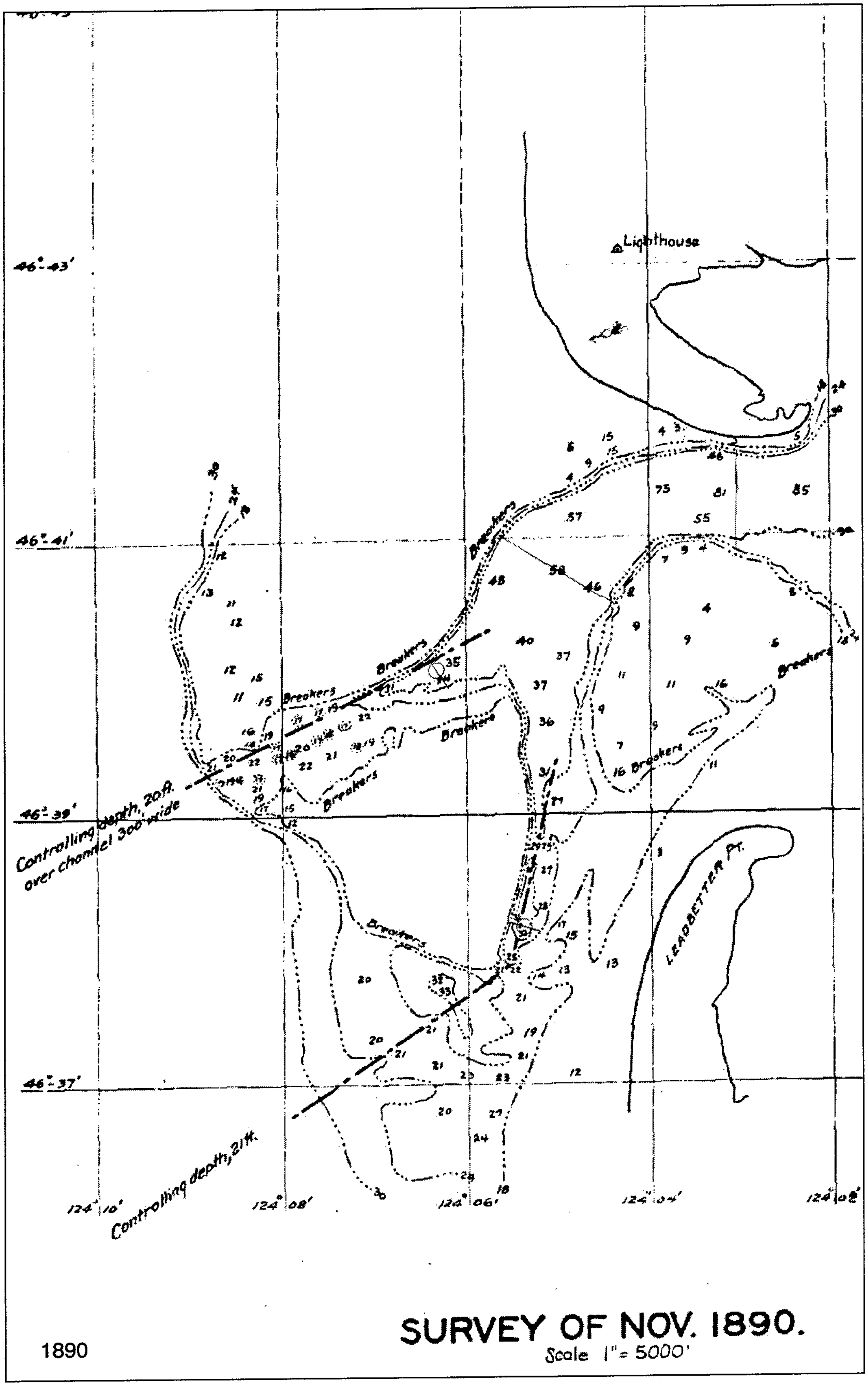
46°
46° 43'
Shore line probably not correct
46° 41'
Controlling depth, 19ft.?
Breakers
46° 39'
Bare at Low tide
Bare at low tide
Leadbetter Pt.
46° 37'
124° 10'
124° 08'
124° 06'
124° 04'
124° 02'
Controlling depth, 25ft.
SURVEY OF 1852.
1852

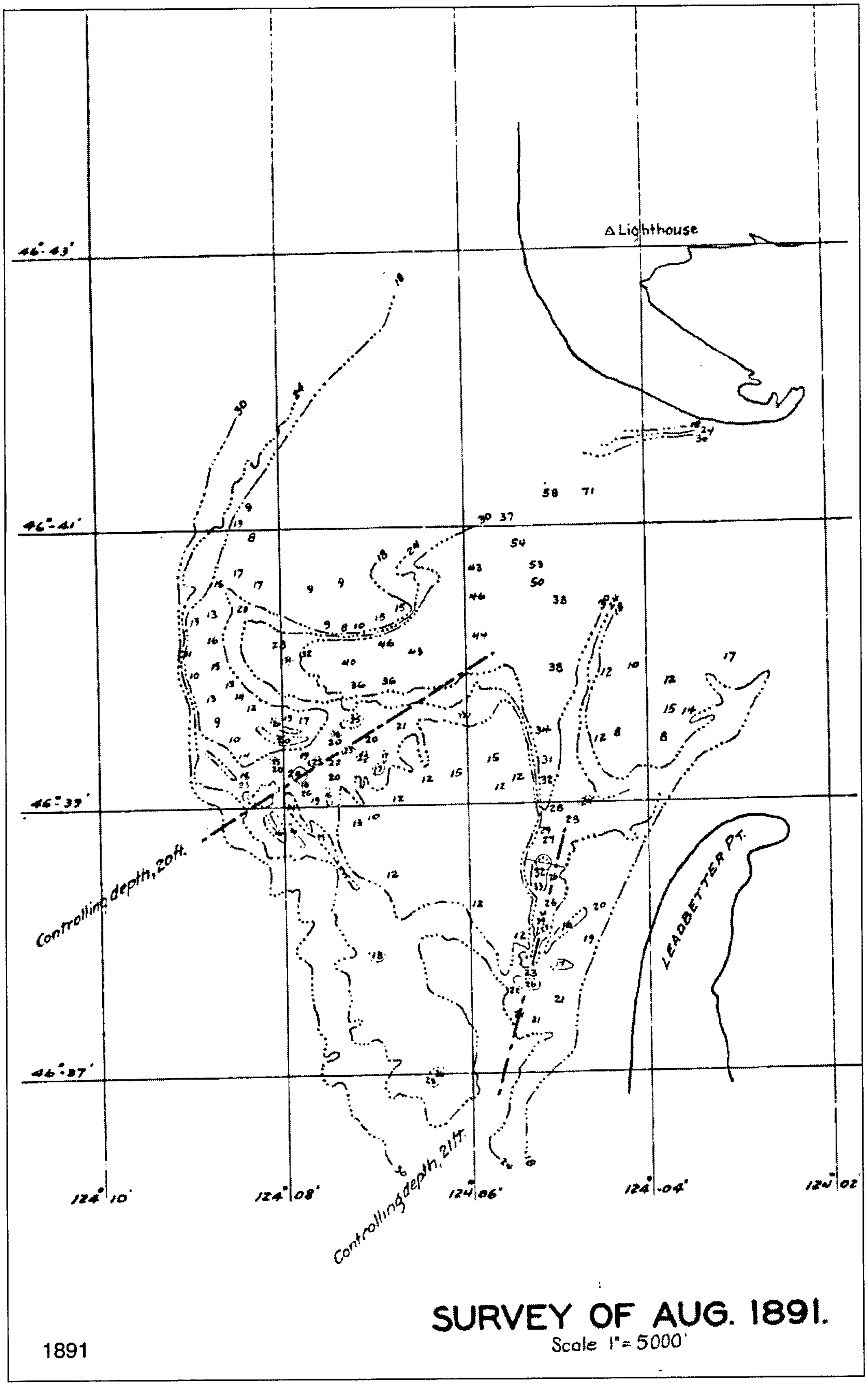


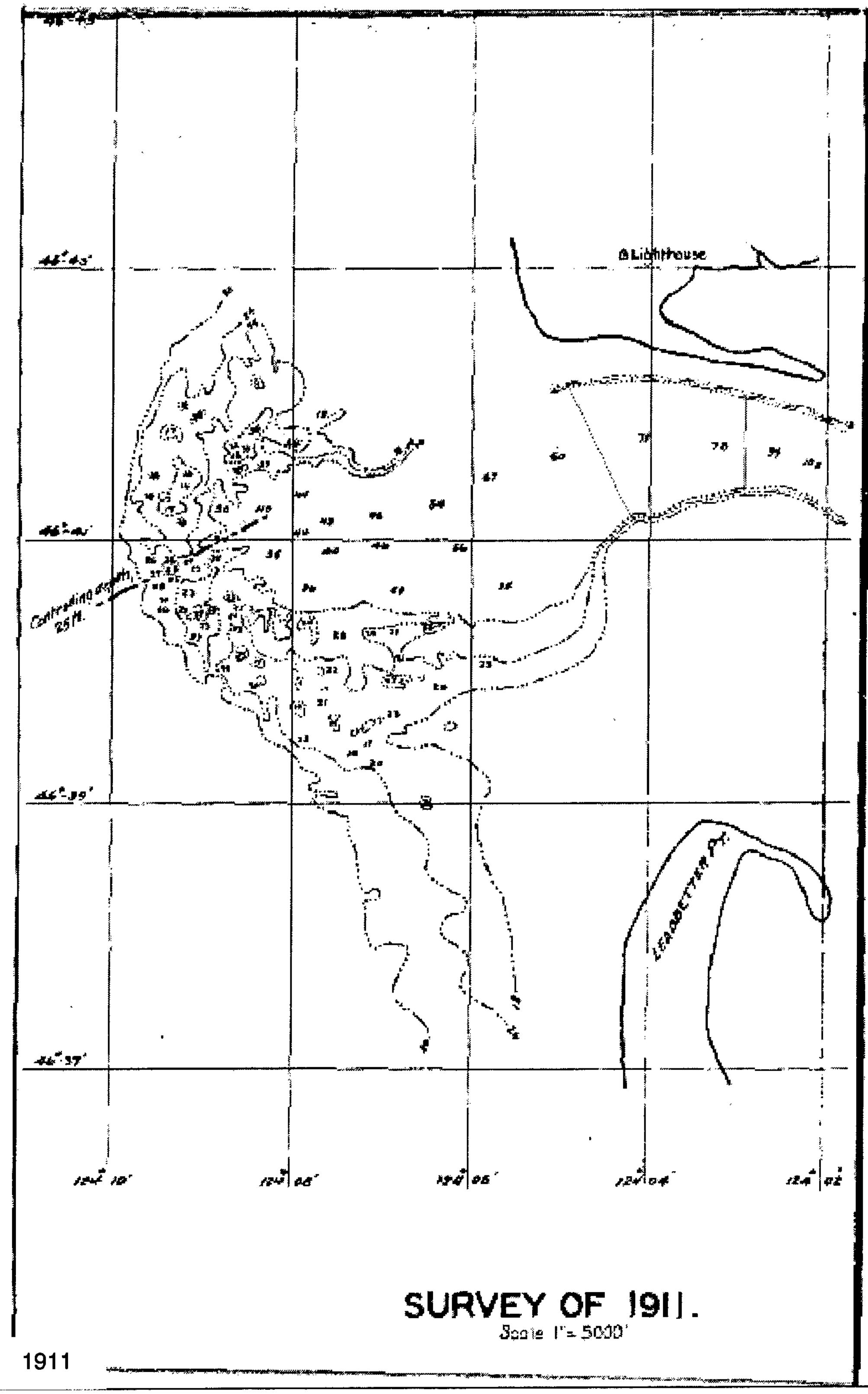
SURVEY OF AUG. 1887.

Scale 1" = 5000'

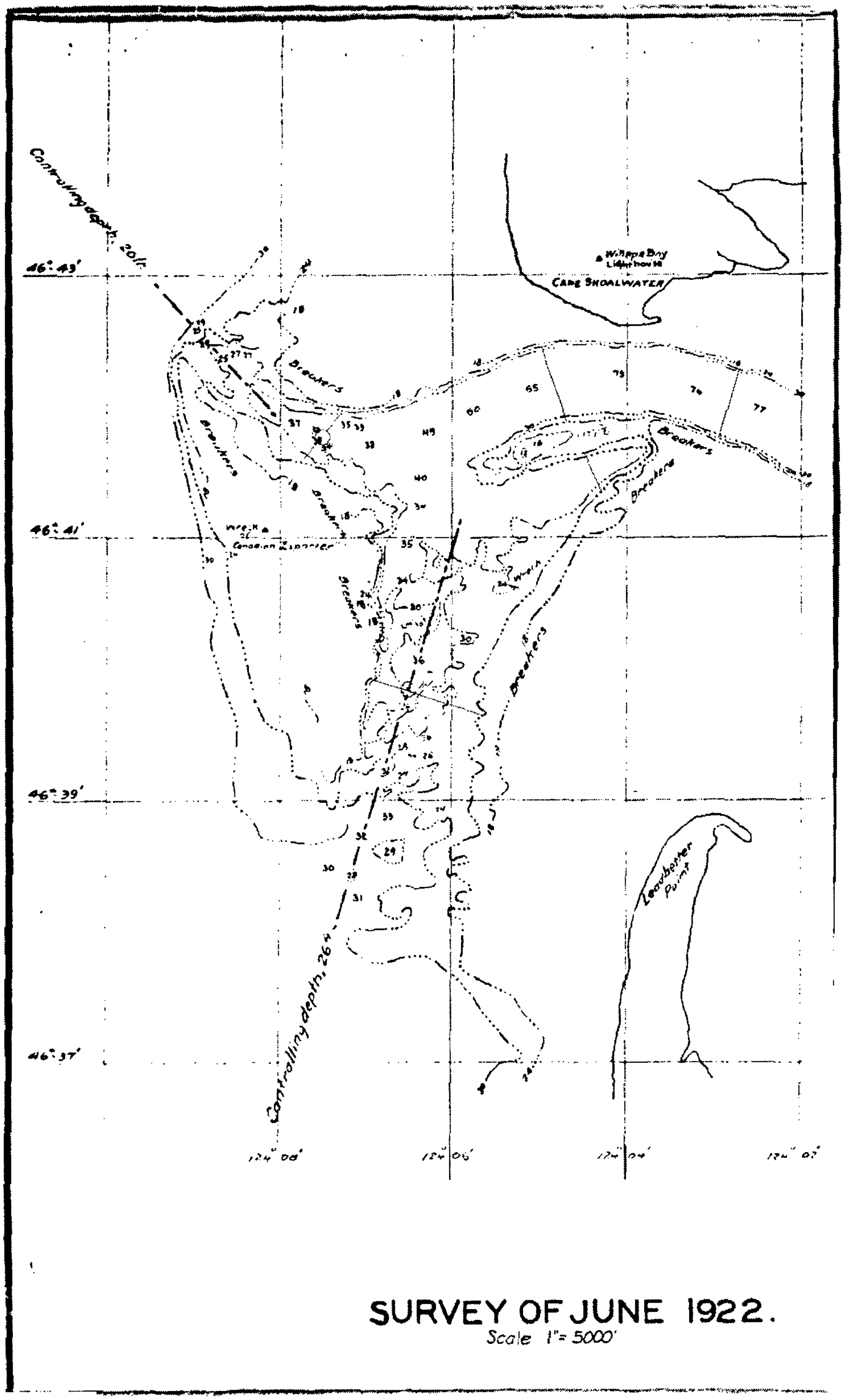
1887


Lighthouse
LEADBETTER PT.
Breakers
Controlling depth, 20 ft. over channel 300' wide
Controlling depth, 21 ft.
46°-43'
46°-41'
46°-39'
46°-37'
124°10'
124°08'
124°06'
124°04'
124°02'
SURVEY OF NOV. 1890.
Scale 1" = 5000'
1890


Lighthouse
46°-43'
46°-41'
46°-39'
46°-37'
LEADBETTER PT.
Controlling depth, 20ft.
Controlling depth, 21ft.
124° 10'
124° 08'
124° 06'
124° -04'
124° 02'
SURVEY OF AUG. 1891.
Scale 1" = 5000'
1891

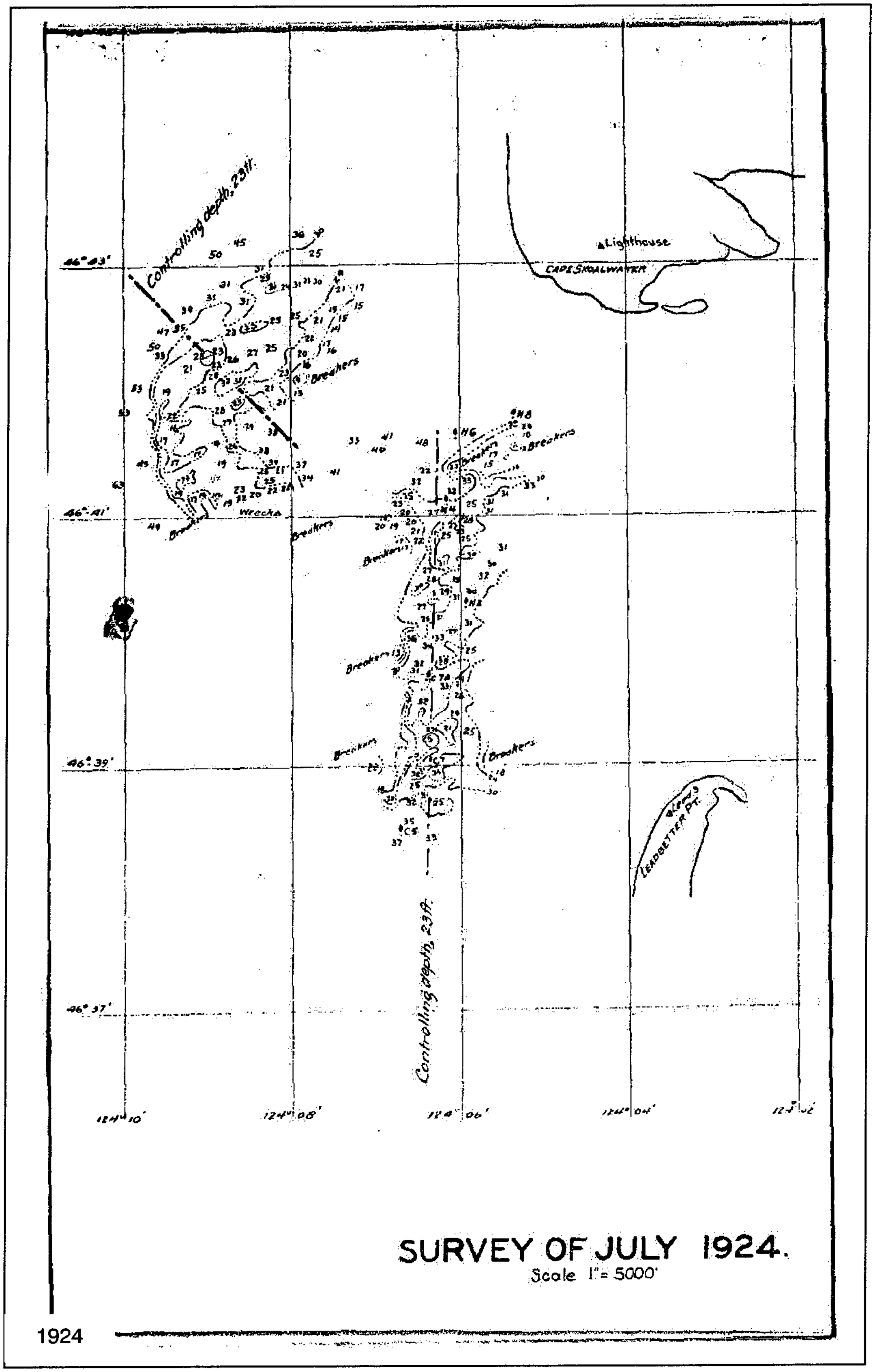

Lighthouse
LEADBETTER PT.
Controlling depth 25 ft.
124° 10'
124° 08'
124° 06'
124° 04'
124° 02'
46° 43'
46° 41'
46° 39'
46° 37'
SURVEY OF 1911.
Scale 1"= 5000'

1911



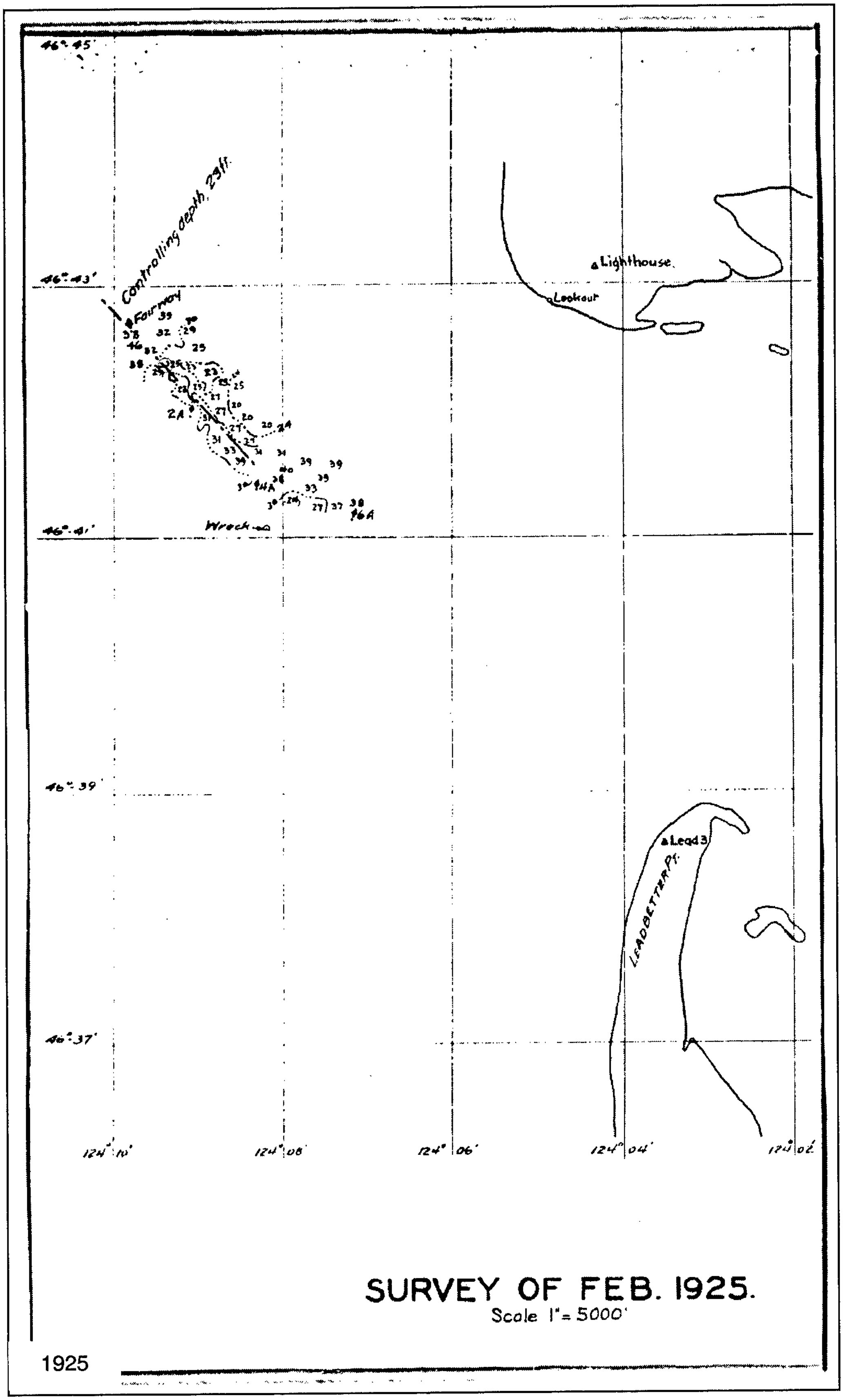
Controlling depth, 20 ft.
46° 43'
46° 41'
46° 39'
46° 37'
Willapa Bay Lighthouse
Cape Shoalwater
Breakers
Wreck
Leadbetter Point
Controlling depth, 26 ft.
124° 08'
124° 06'
124° 04'
124° 02'
SURVEY OF JUNE 1922.
Scale 1" = 5000'

1922

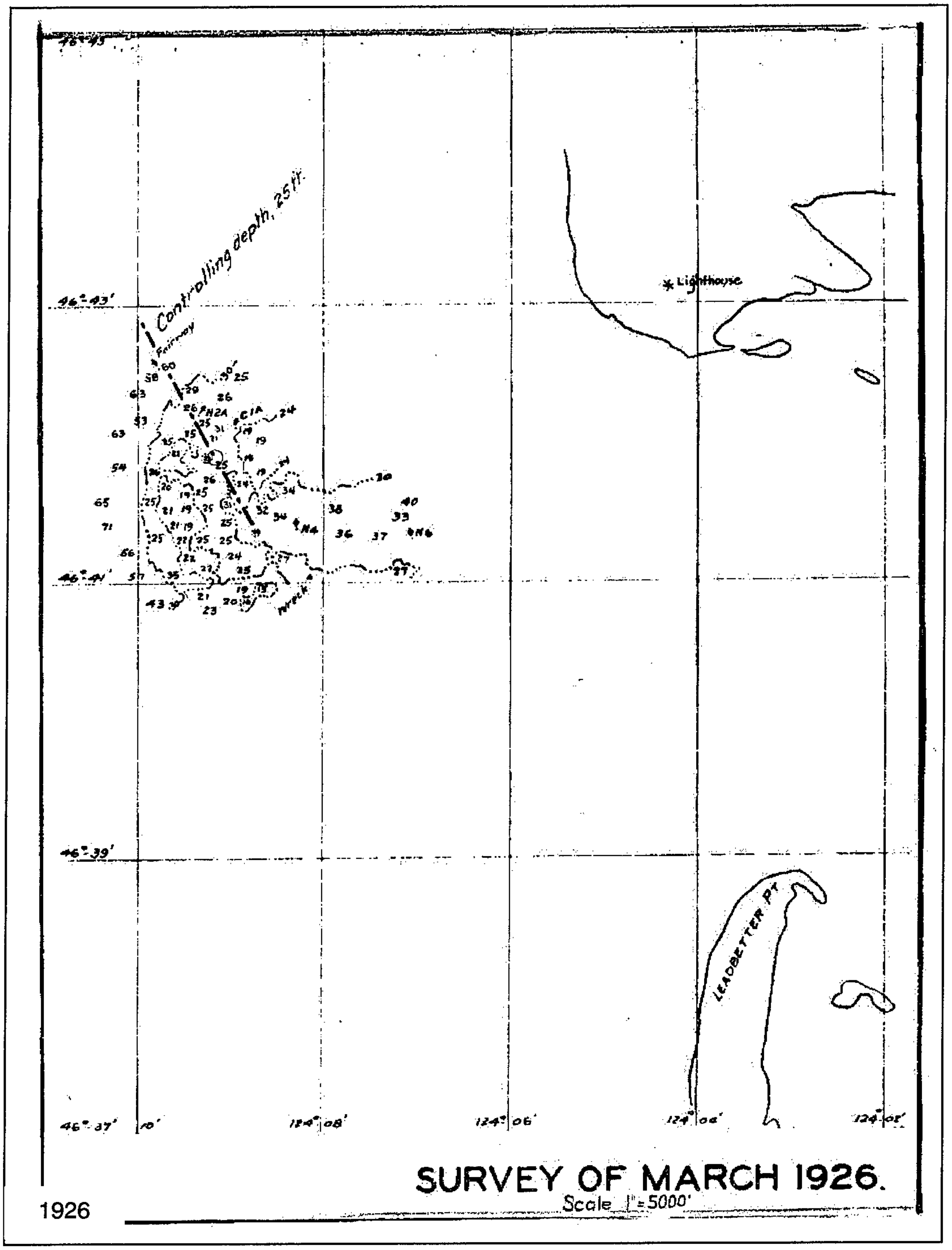


Controlling depth, 23 ft.
Lighthouse
CAPE SHOALWATER
Breakers
Wrecks
LEADBETTER PT.
46° 43'
46° 41'
46° 39'
46° 37'
124° 10'
124° 08'
124° 06'
124° 04'
124° 02'
SURVEY OF JULY 1924.
Scale 1" = 5000'

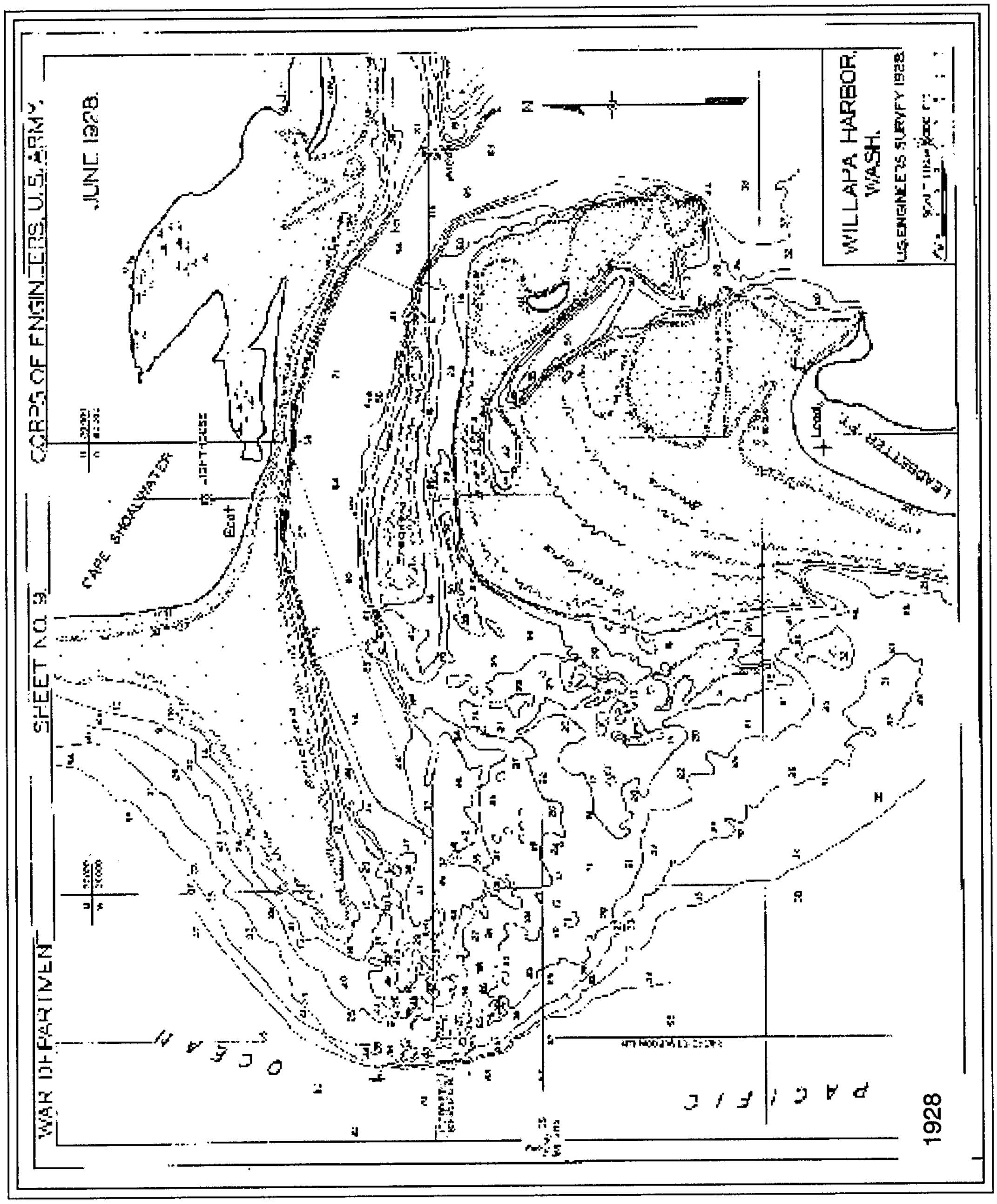
1924

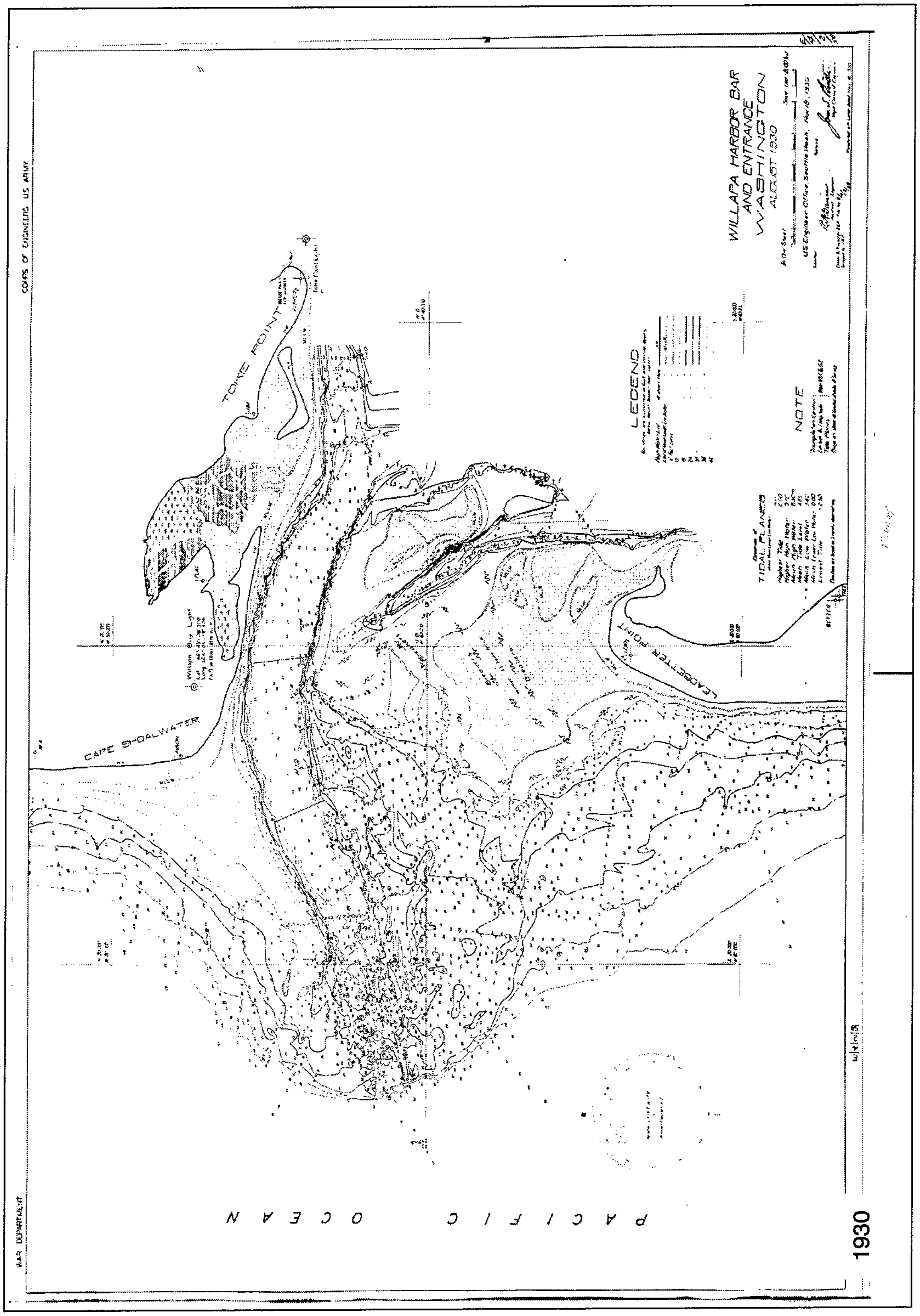


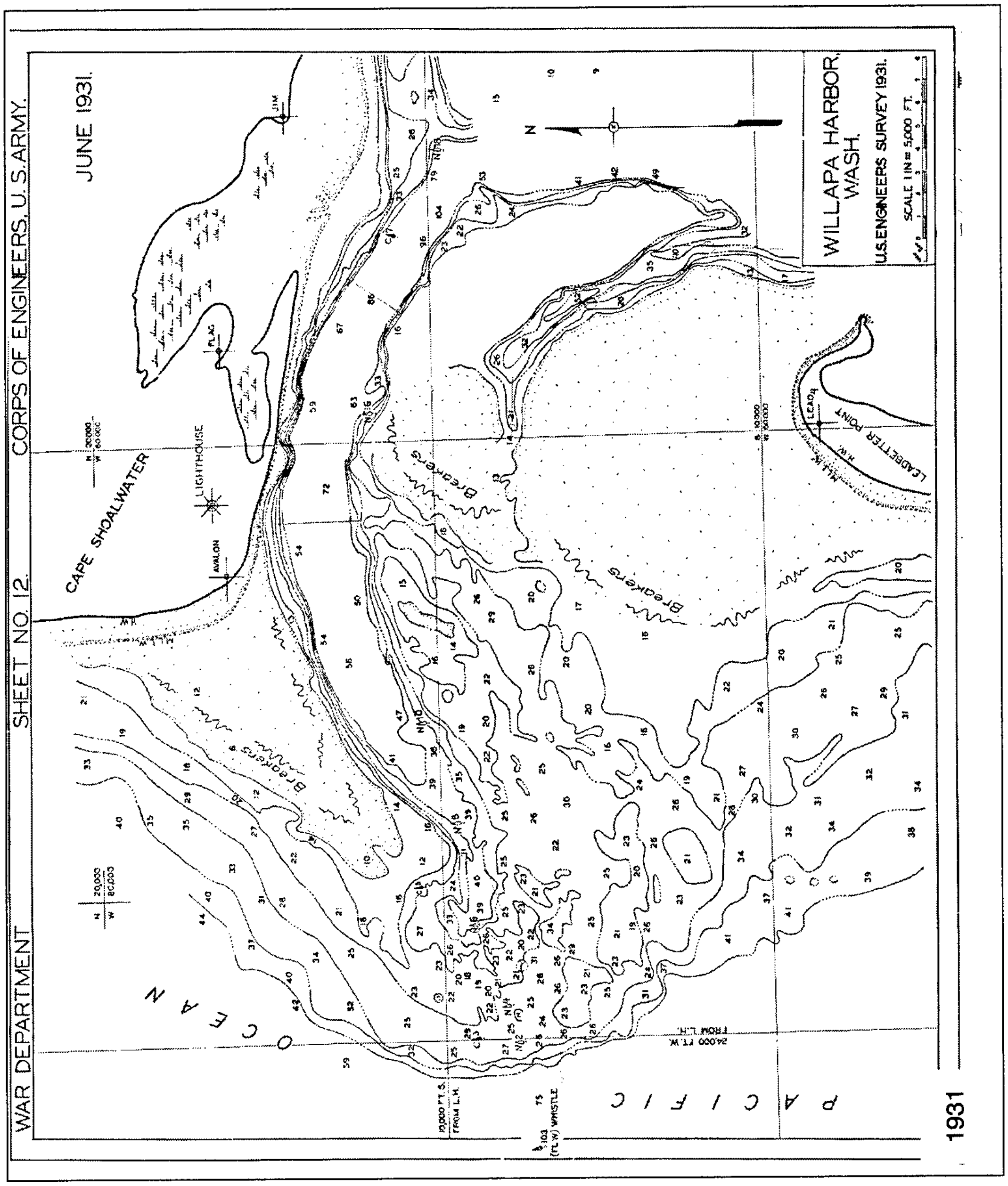
46° 45'
46° 43'
46° 41'
46° 39'
46° 37'
Controlling depth 23ft.
Fairway
Lighthouse
Lookout
Wreck
Lead 3
LEADBETTER PT.
124° 10'
124° 08'
124° 06'
124° 04'
124° 02'
SURVEY OF FEB. 1925.
Scale 1"= 5000'
1925

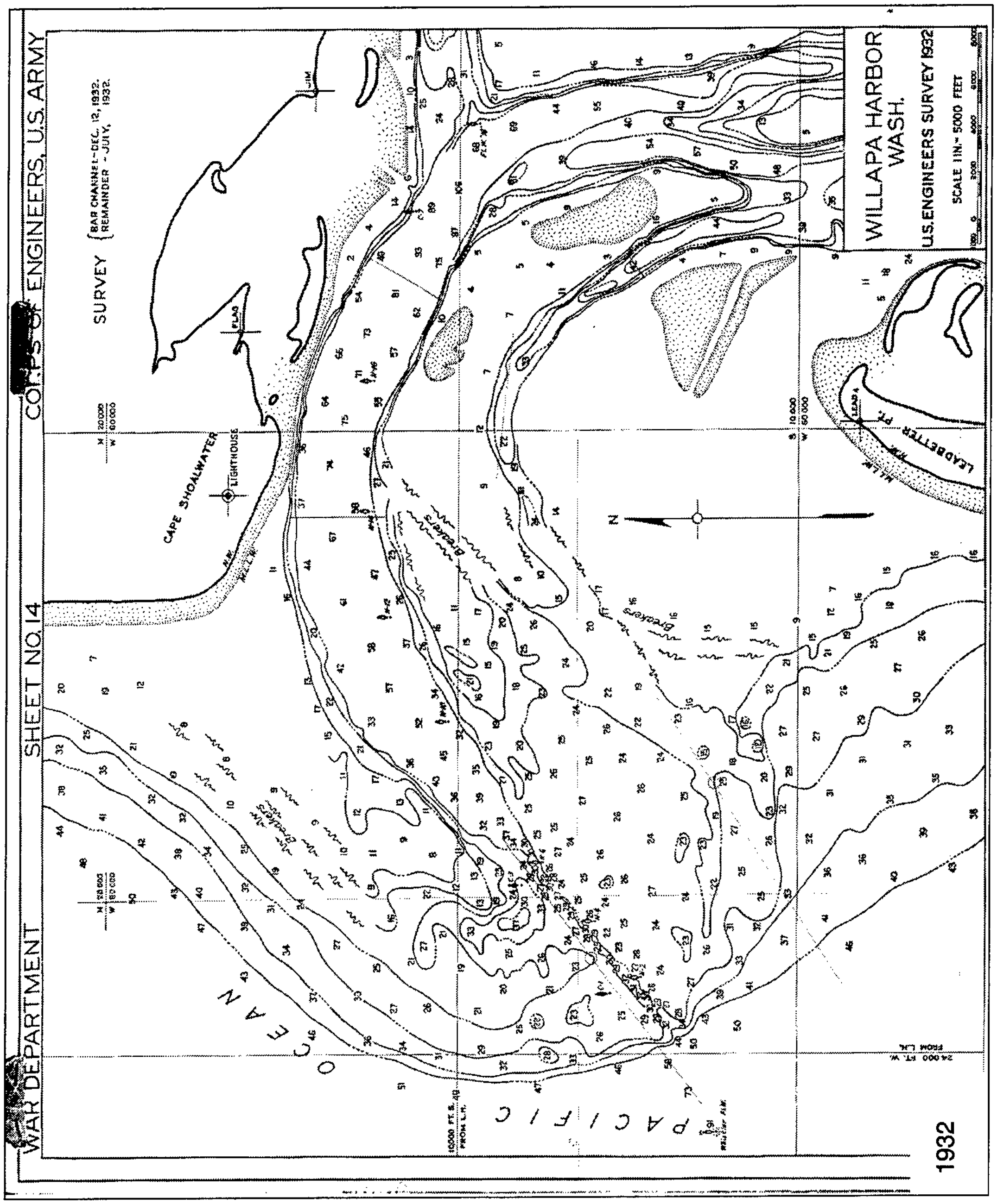

Controlling depth, 25 ft.
Fairway
Wreck
Lighthouse
LEADBETTER PT
46° 43'
46° 41'
46° 39'
46° 37'
10'
124° 08'
124° 06'
124° 04'
124° 02'
SURVEY OF MARCH 1926.
Scale 1" = 5000'

1926

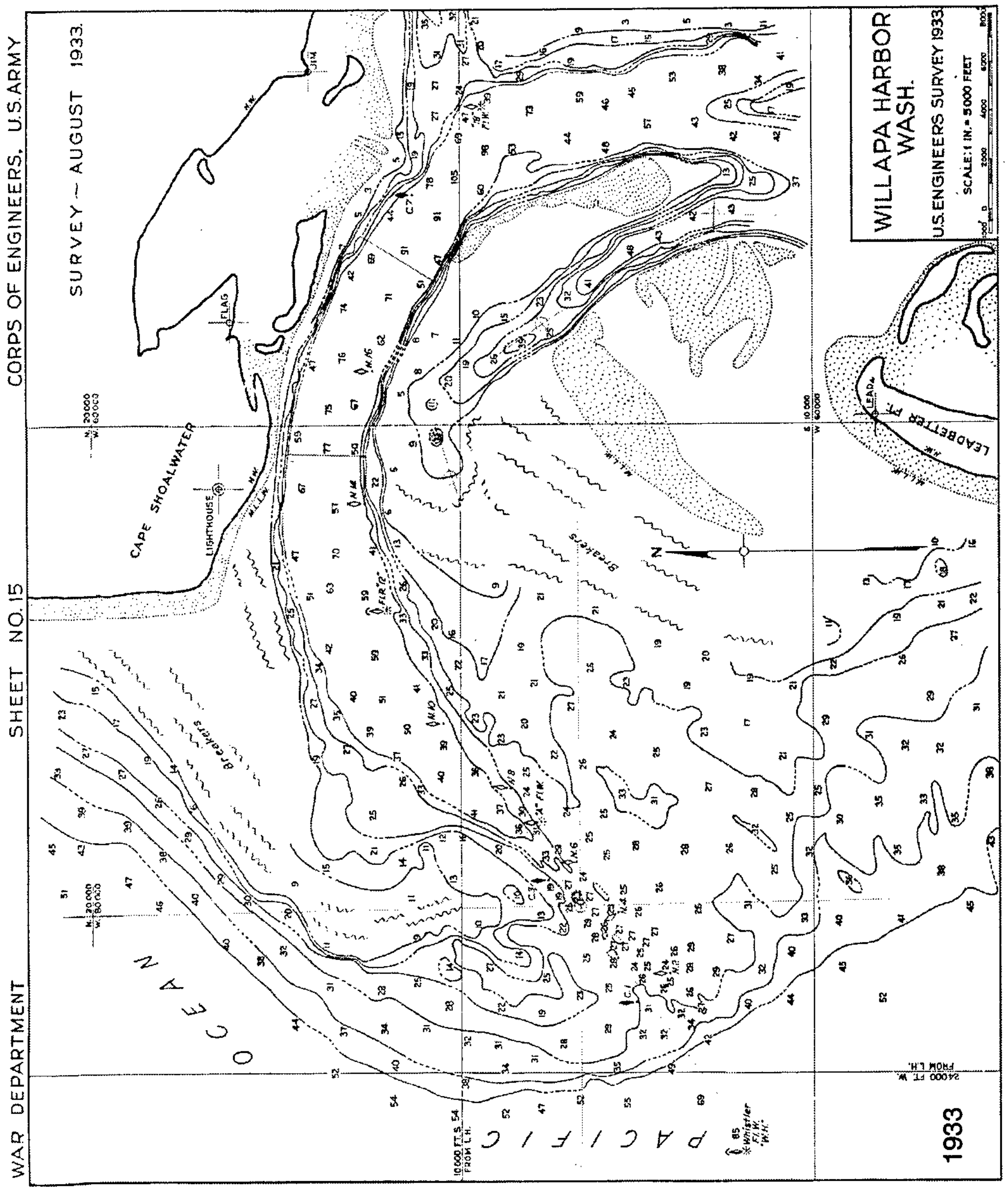

WAR DEPARTMENT
SHEET NO. 9
CORPS OF ENGINEERS, U.S. ARMY
JUNE 1928.
CAPE SHOALWATER
LIGHTHOUSE
WILLAPA HARBOR, WASH.
U.S. ENGINEER'S SURVEY 1928.
LEADBETTER PT.
OCEAN
PACIFIC
1928

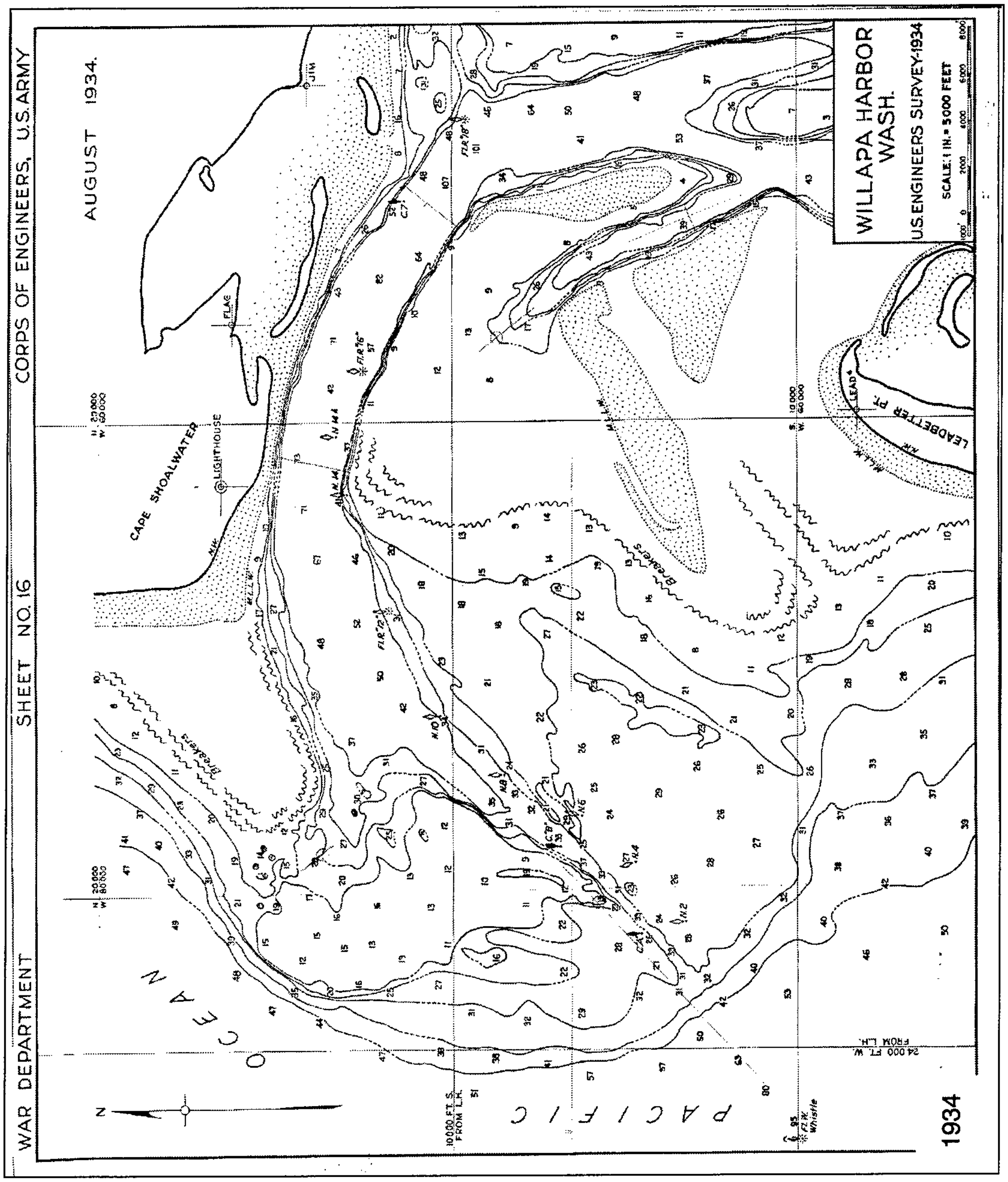

WILLAPA HARBOR BAR
AND ENTRANCE
WASHINGTON
AUGUST 1930
CORPS OF ENGINEERS US ARMY
WAR DEPARTMENT
TOKE POINT
CAPE SHOALWATER
LEADBETTER POINT
LEGEND
NOTE
TIDAL PLANES
PACIFIC OCEAN
1930

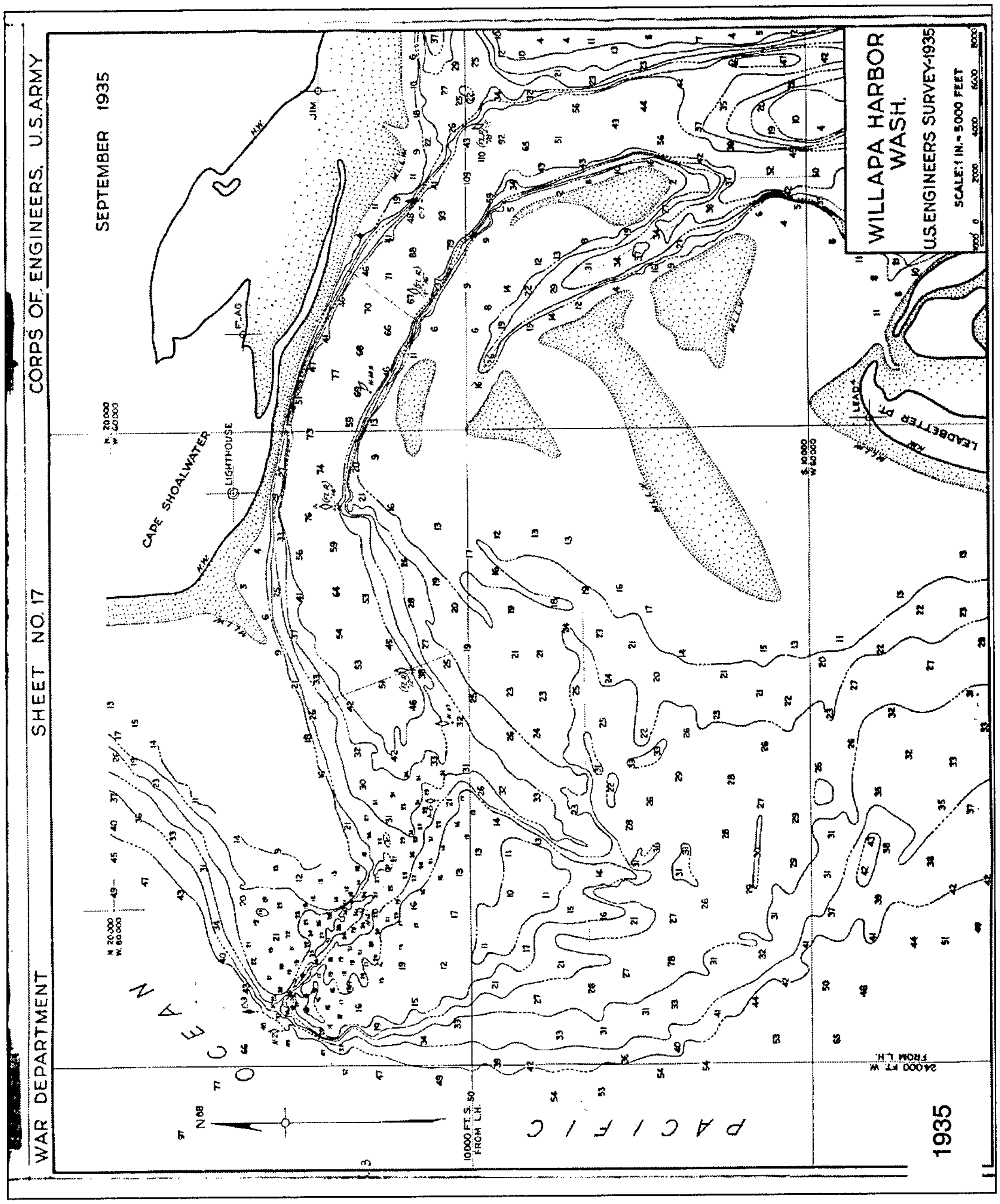

WAR DEPARTMENT
SHEET NO. 12.
CORPS OF ENGINEERS, U.S. ARMY.
JUNE 1931.
WILLAPA HARBOR, WASH.
U.S. ENGINEERS SURVEY 1931.
SCALE 1 IN = 5,000 FT.
CAPE SHOALWATER
LIGHTHOUSE
AVALON
FLAG
Breakers
LEADBETTER POINT
PACIFIC
OCEAN
N
1931



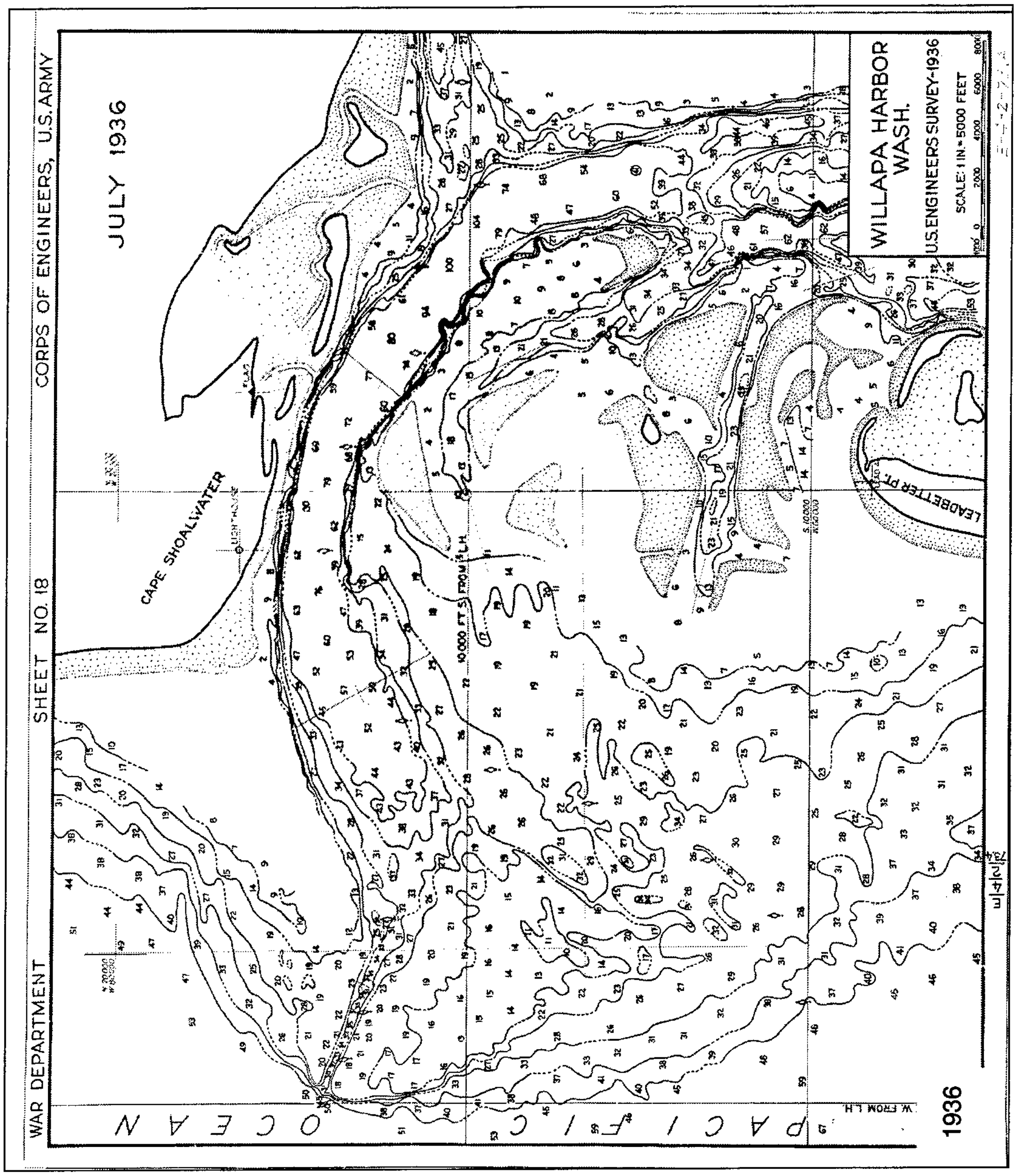
WAR DEPARTMENT
SHEET NO.14
ENGINEERS, U.S. ARMY
SURVEY
BAR CHANNEL - DEC. 12, 1932.
REMAINDER - JULY, 1932.
CAPE SHOALWATER
LIGHTHOUSE
LEADBETTER PT.
Breakers
PACIFIC
OCEAN
WILLAPA HARBOR
WASH.
U.S. ENGINEERS SURVEY 1932
SCALE 1 IN. = 5000 FEET
1932


WAR DEPARTMENT
SHEET NO.15
CORPS OF ENGINEERS, U.S.ARMY
SURVEY — AUGUST 1933.
CAPE SHOALWATER
LIGHTHOUSE
FLAG
Breakers
LEADBETTER PT.
OCEAN
PACIFIC
WILLAPA HARBOR
WASH.
U.S.ENGINEERS SURVEY 1933
SCALE: 1 IN. = 5000 FEET
1933

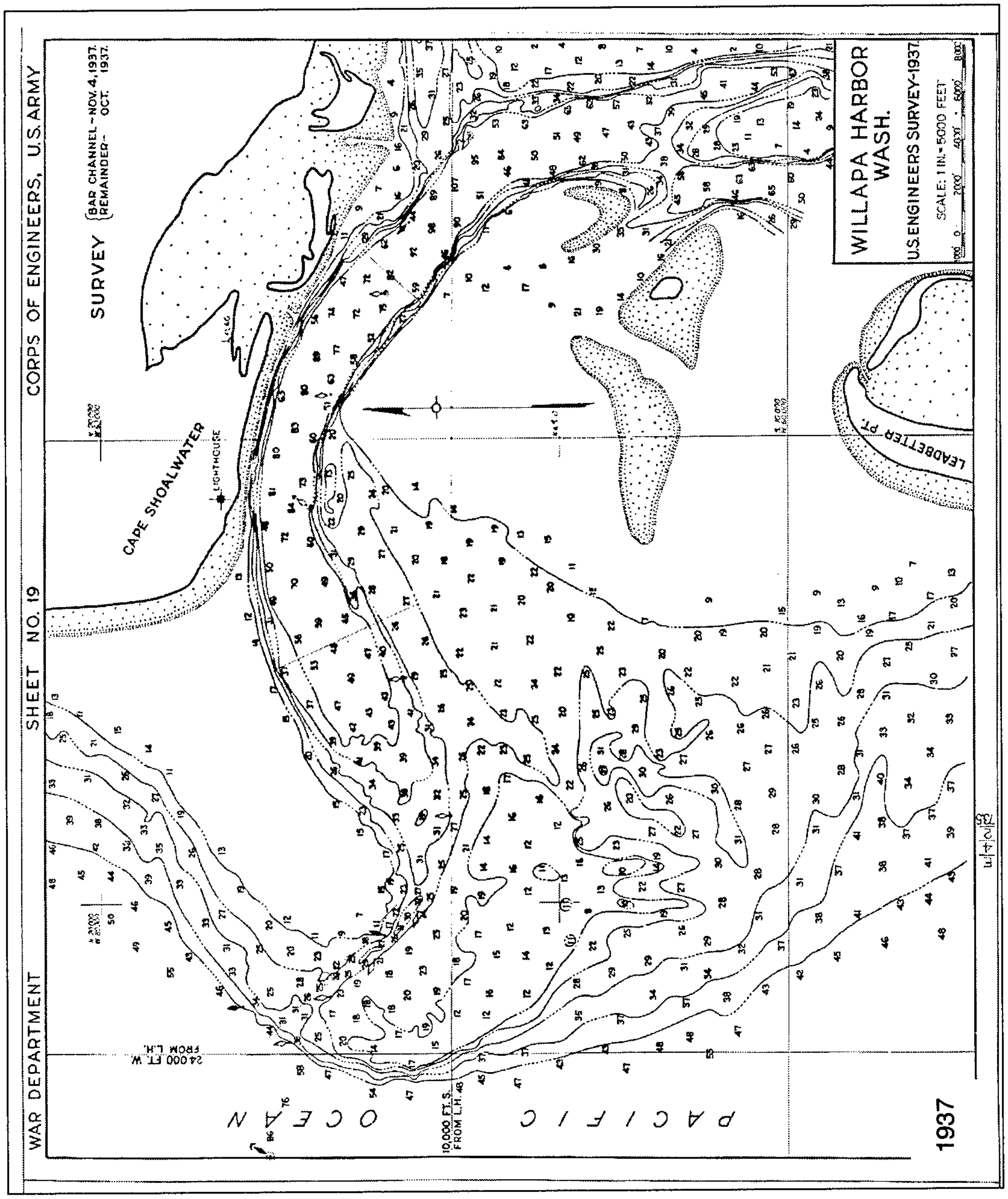

WAR DEPARTMENT
SHEET NO. 16
CORPS OF ENGINEERS, U.S. ARMY
AUGUST 1934.
WILLAPA HARBOR
WASH.
U.S. ENGINEERS SURVEY-1934
SCALE: 1 IN. = 5000 FEET
CAPE SHOALWATER
LIGHTHOUSE
FLAG
Breakers
LEADBETTER PT.
OCEAN
PACIFIC
1934

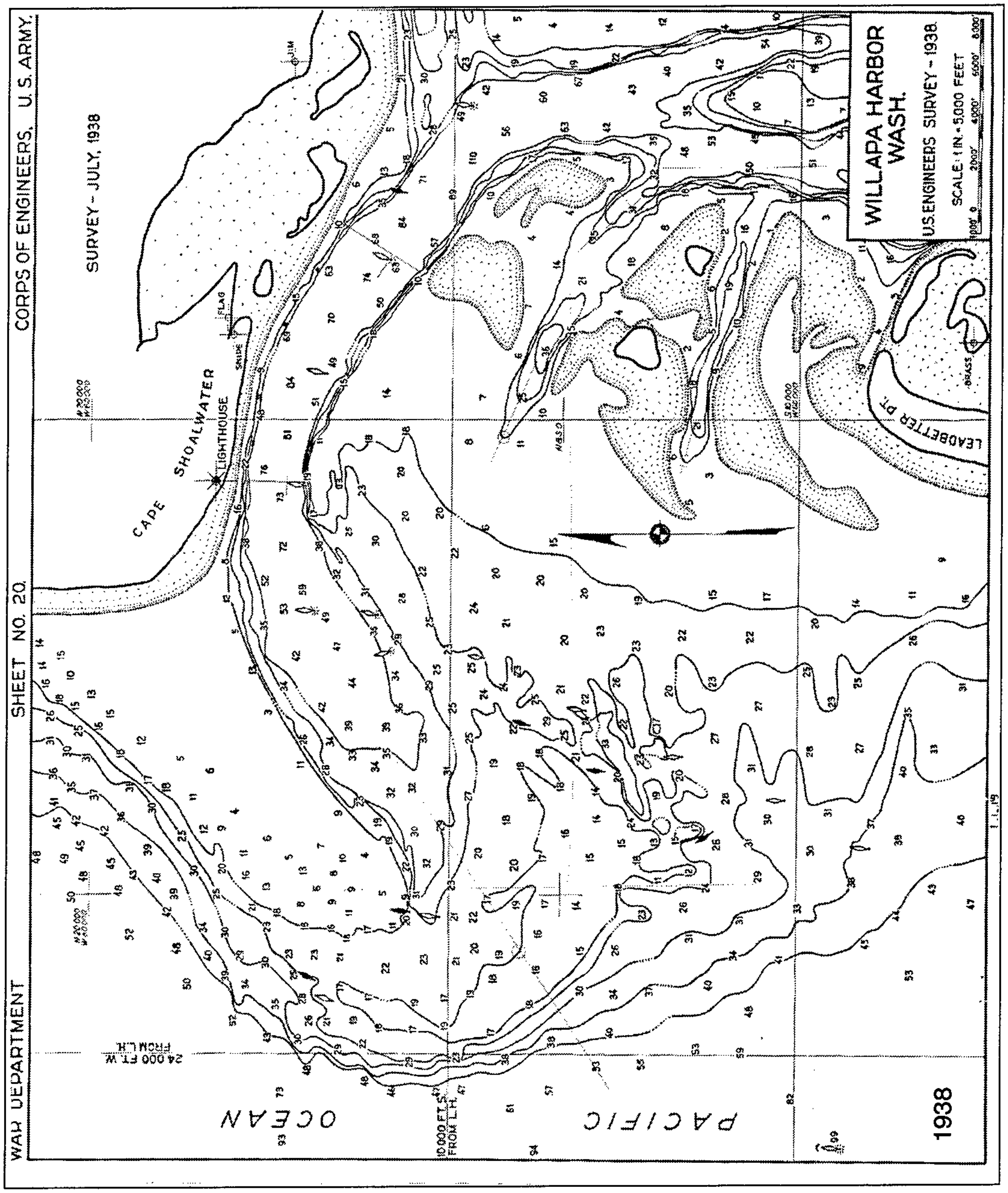


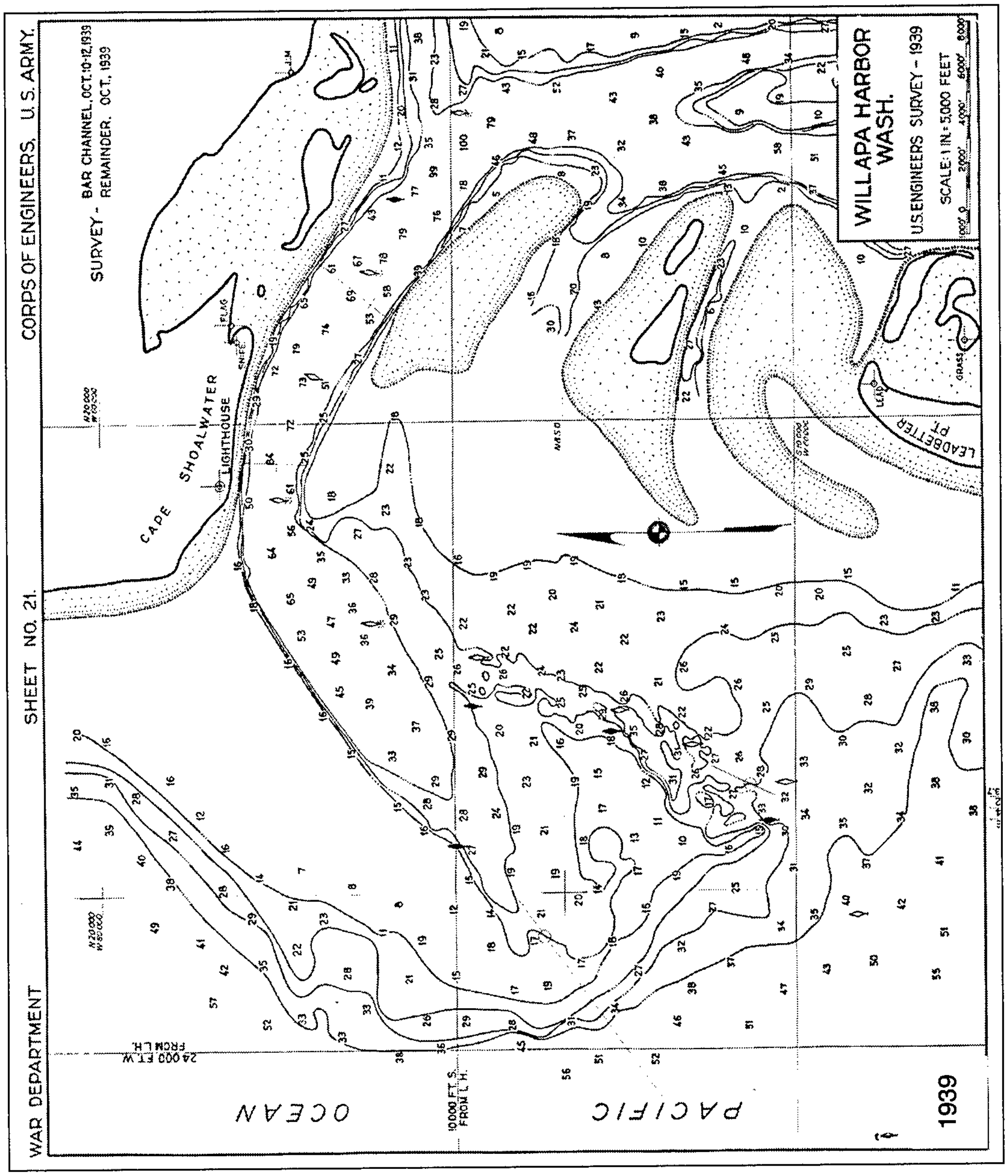
WAR DEPARTMENT
SHEET NO.17
CORPS OF ENGINEERS, U.S.ARMY
SEPTEMBER 1935
CAPE SHOALWATER
LIGHTHOUSE
FLAG
JIM
LEADBETTER PT.
PACIFIC
OCEAN
WILLAPA HARBOR
WASH.
U.S.ENGINEERS SURVEY-1935
SCALE:1 IN.=5000 FEET
1935

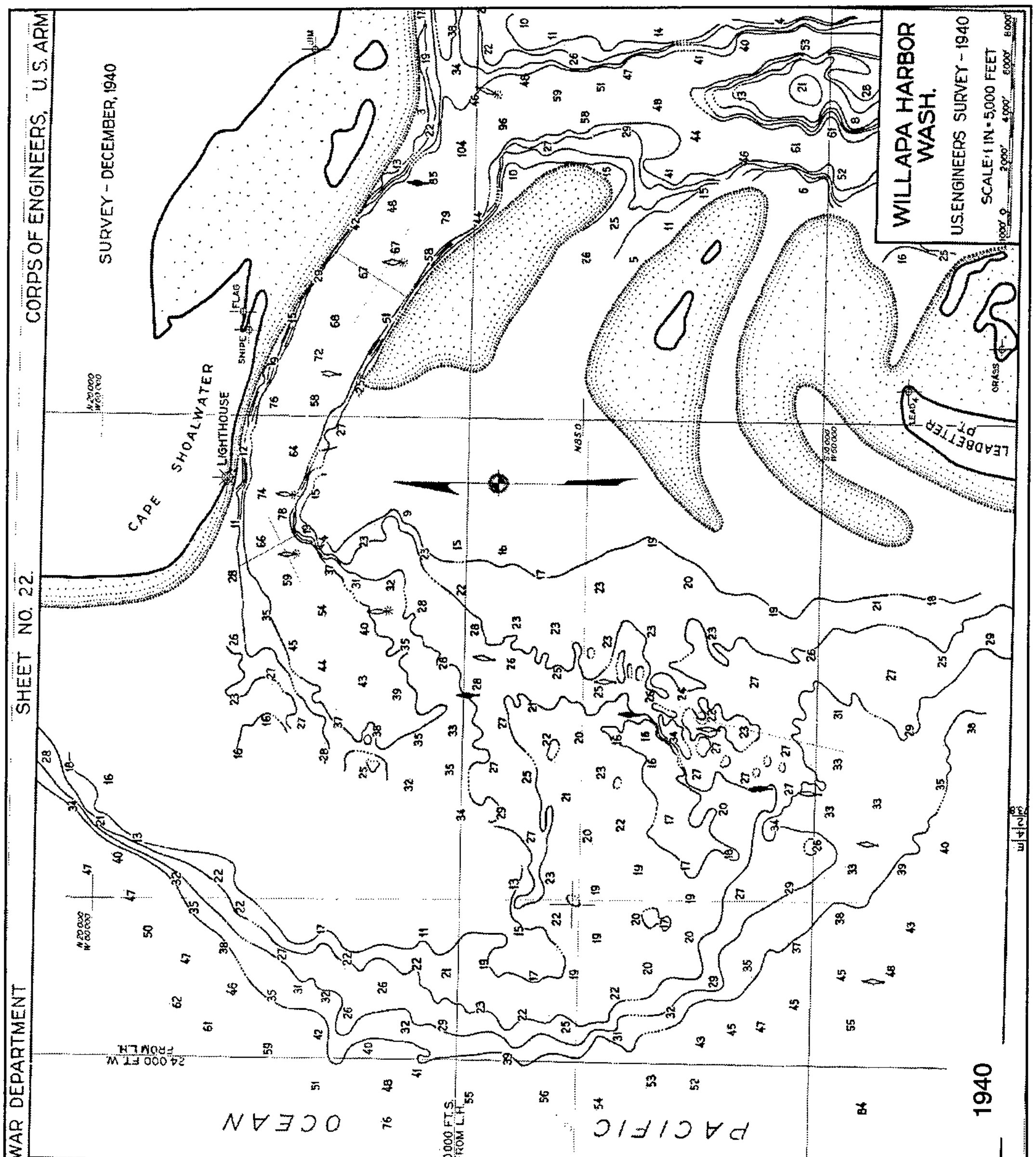


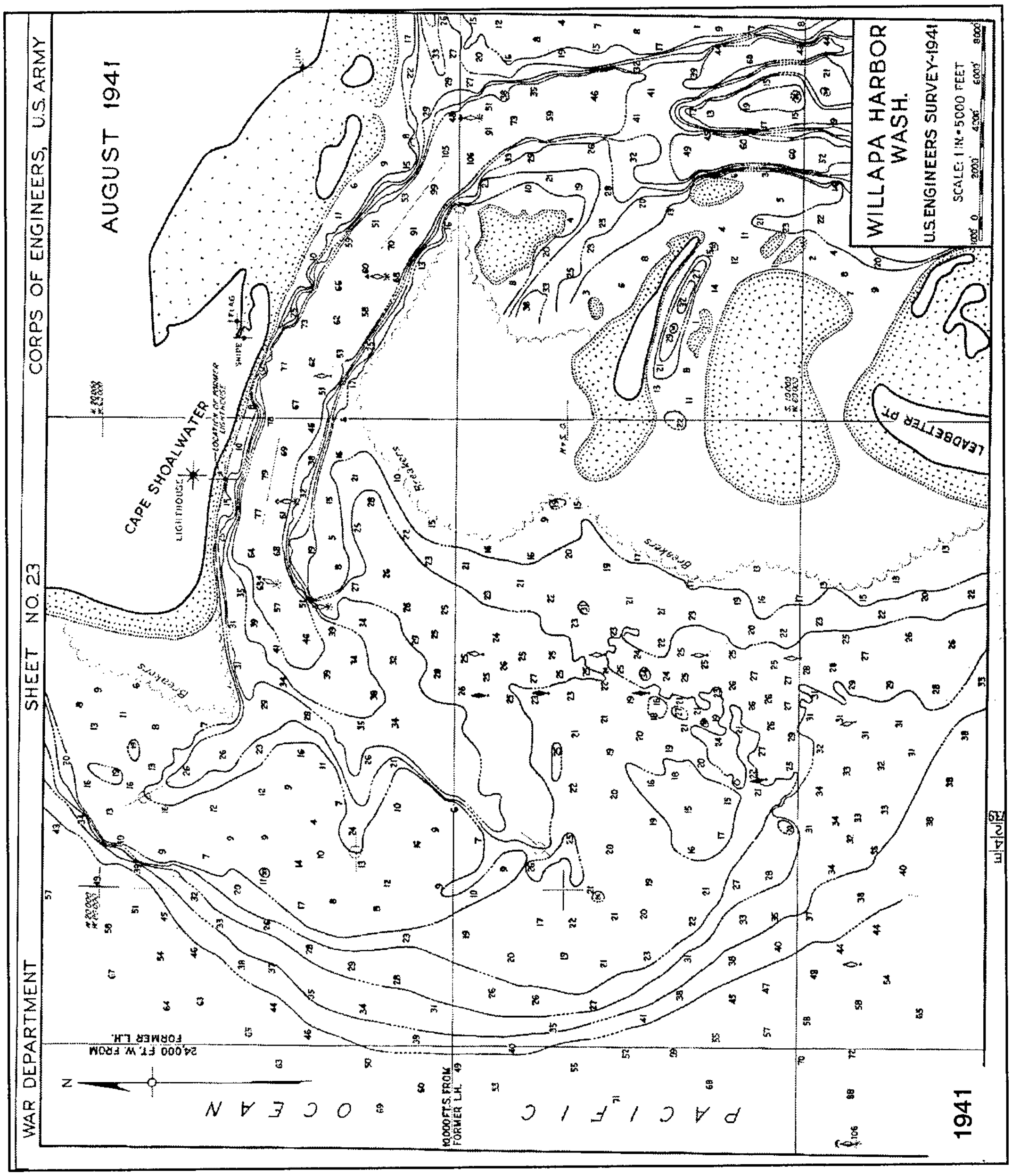
WAR DEPARTMENT
SHEET NO. 18
CORPS OF ENGINEERS, U.S. ARMY
JULY 1936
CAPE SHOALWATER
WILLAPA HARBOR
WASH.
U.S. ENGINEERS SURVEY-1936
SCALE: 1 IN. = 5000 FEET
PACIFIC OCEAN
LEADBETTER PT.
10,000 FT. S. FROM L.H.
1936

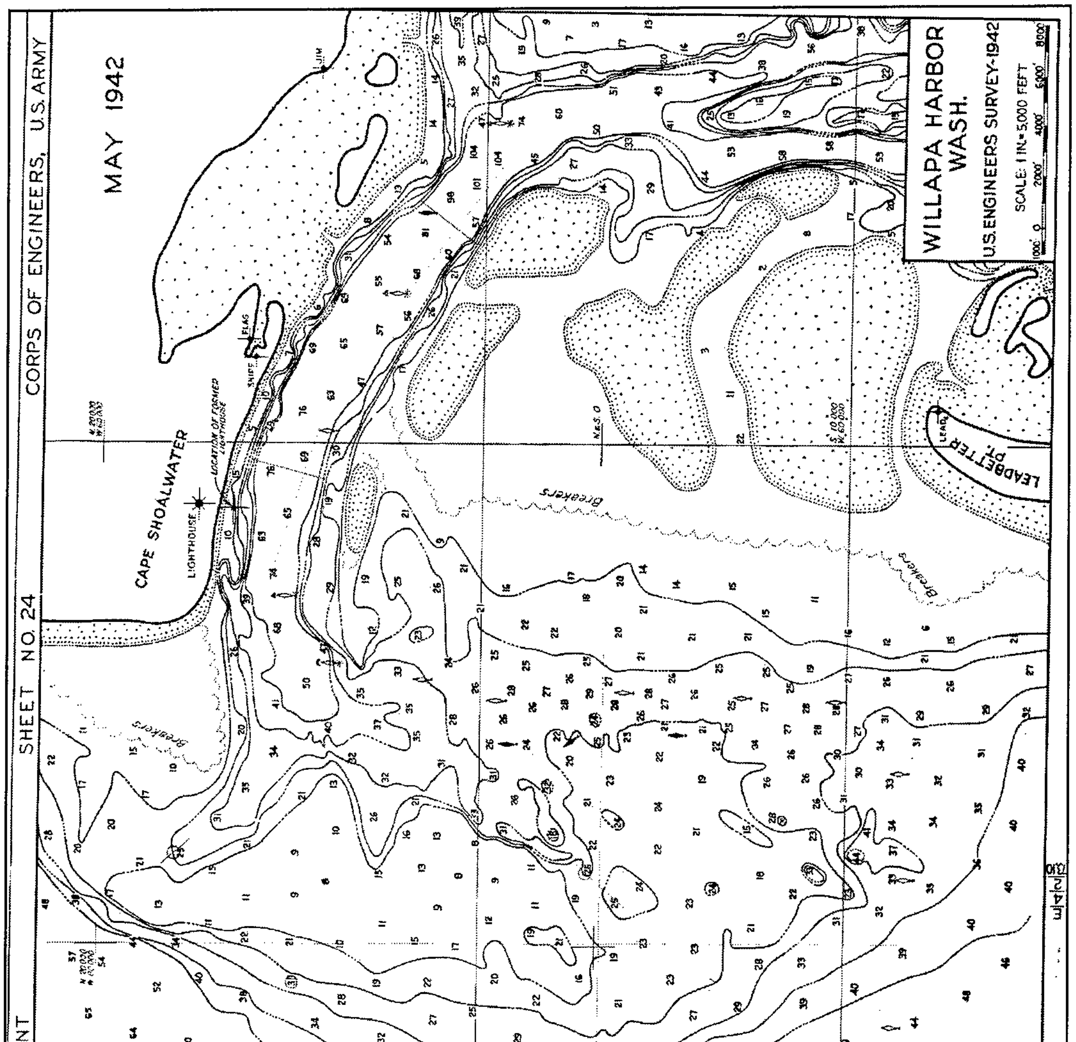

WAR DEPARTMENT
SHEET NO. 19
CORPS OF ENGINEERS, U.S. ARMY
SURVEY {BAR CHANNEL - NOV. 4, 1937. REMAINDER - OCT. 1937.
CAPE SHOALWATER
LIGHTHOUSE
LEADBETTER PT.
WILLAPA HARBOR WASH.
U.S. ENGINEERS SURVEY-1937.
SCALE: 1 IN. = 5000 FEET
PACIFIC OCEAN
10,000 FT. S. FROM L.H.
24000 FT. W. FROM L.H.
1937

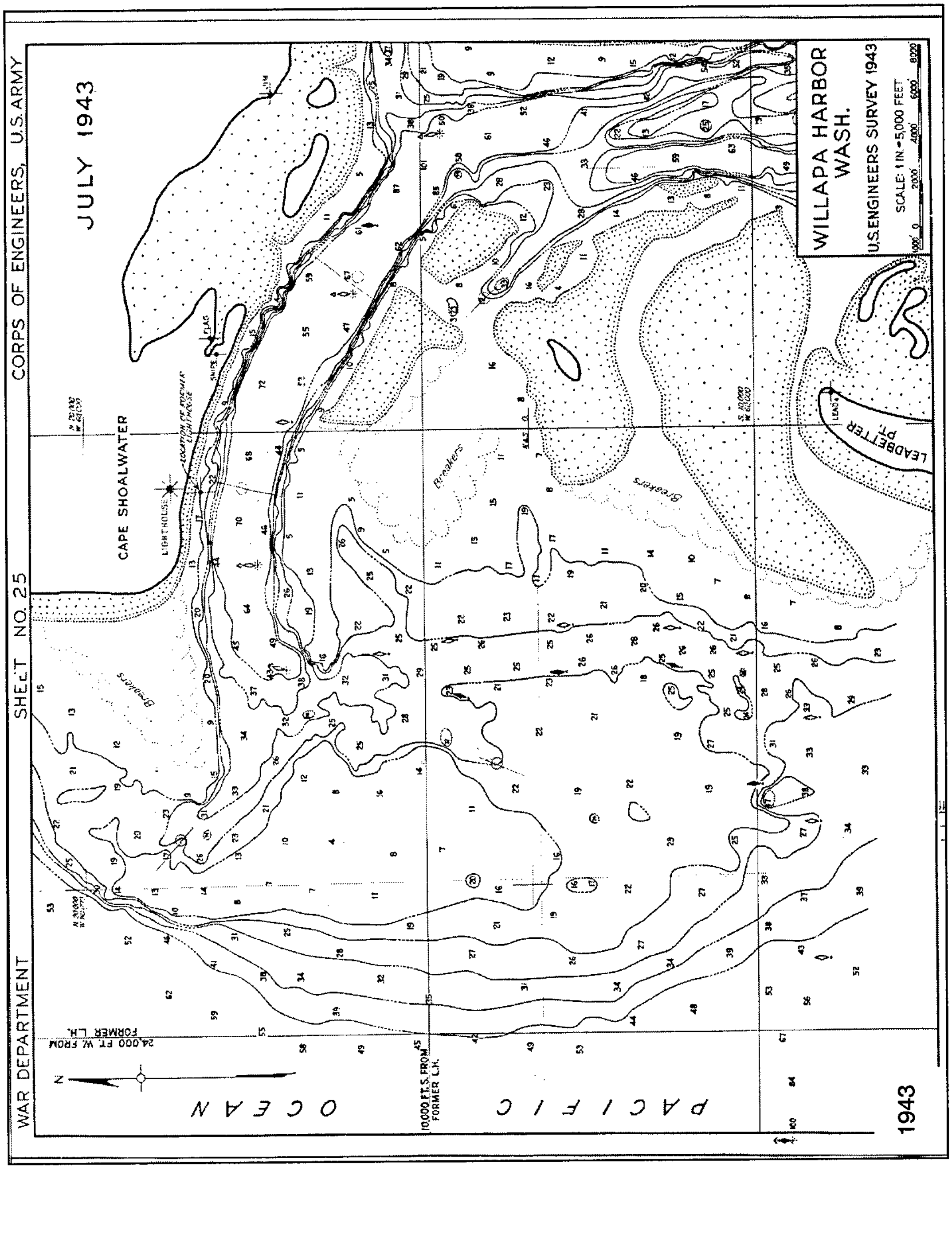

WAR DEPARTMENT
SHEET NO. 20
CORPS OF ENGINEERS, U.S. ARMY.
SURVEY - JULY, 1938
CAPE SHOALWATER
LIGHTHOUSE
LEADBETTER PT.
OCEAN
PACIFIC
WILLAPA HARBOR
WASH.
U.S. ENGINEERS SURVEY - 1938.
SCALE: 1 IN. = 5,000 FEET
1938

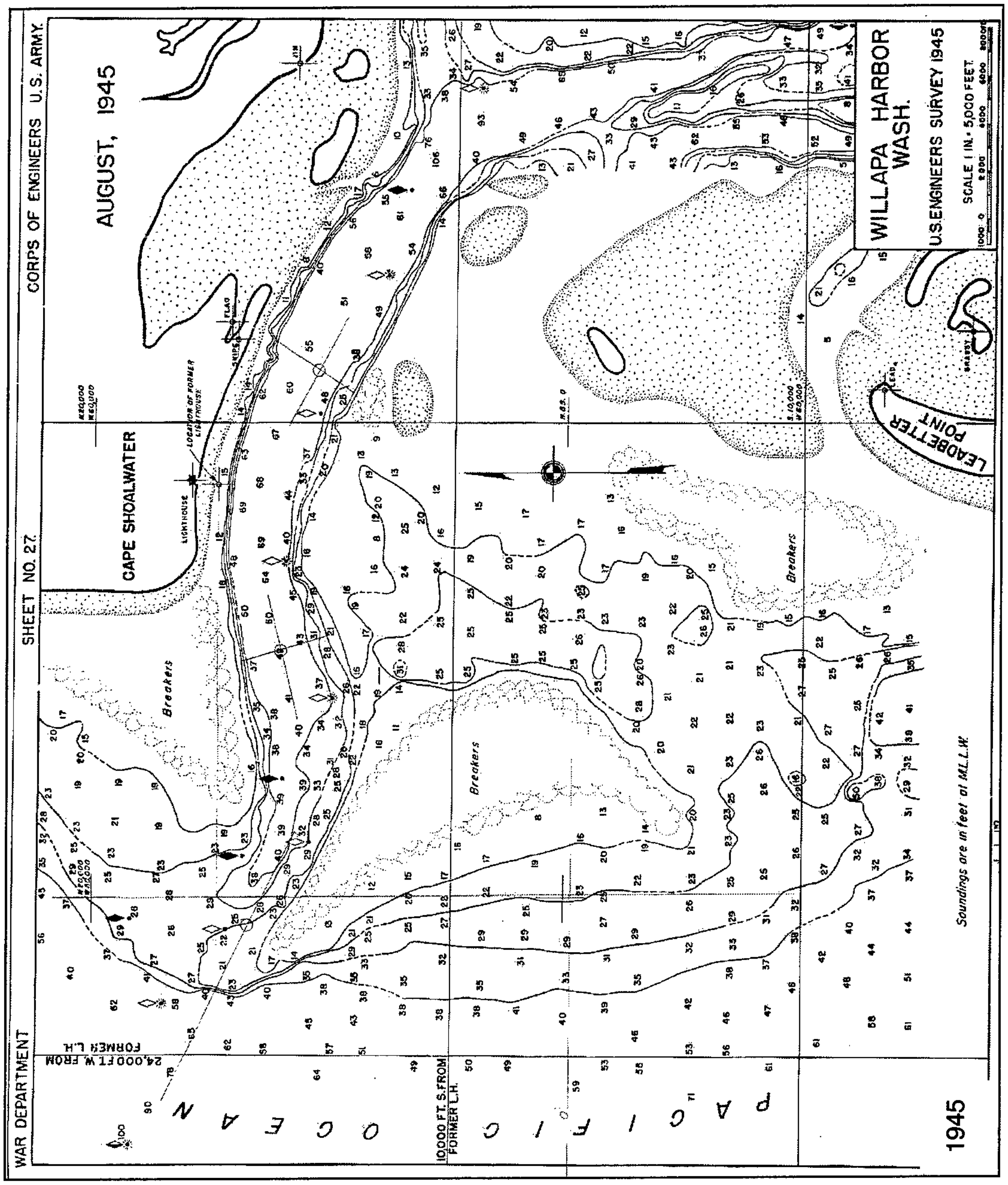

WAR DEPARTMENT
SHEET NO. 21.
CORPS OF ENGINEERS, U.S. ARMY.
SURVEY - BAR CHANNEL, OCT. 10-12, 1939
REMAINDER, OCT. 1939
WILLAPA HARBOR
WASH.
U.S. ENGINEERS SURVEY - 1939
SCALE: 1 IN = 5,000 FEET
CAPE SHOALWATER
LIGHTHOUSE
LEADBETTER PT.
PACIFIC
OCEAN
1939

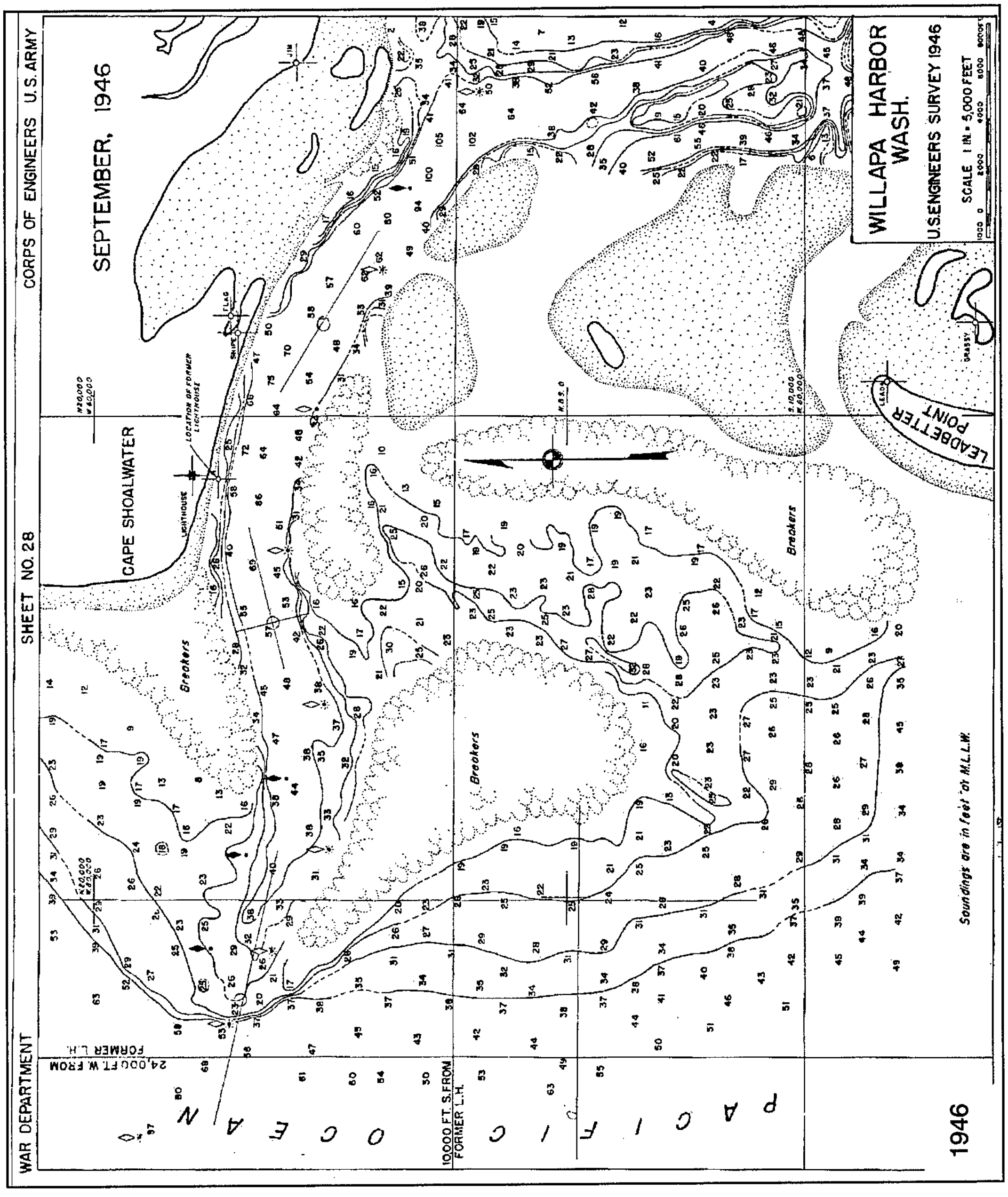

WAR DEPARTMENT
SHEET NO. 22
CORPS OF ENGINEERS, U.S. ARMY
SURVEY - DECEMBER, 1940
CAPE SHOALWATER
LIGHTHOUSE
SNIPE
FLAG
LEADBETTER PT.
GRASS
PACIFIC OCEAN
10,000 FT. S. FROM L.H.
24,000 FT. W. FROM L.H.
WILLAPA HARBOR WASH.
U.S. ENGINEERS SURVEY - 1940
SCALE: 1 IN. = 5,000 FEET
1940


WAR DEPARTMENT
SHEET NO. 23
CORPS OF ENGINEERS, U.S. ARMY
AUGUST 1941
CAPE SHOALWATER
LIGHTHOUSE
LEADBETTER PT.
PACIFIC OCEAN
WILLAPA HARBOR
WASH.
U.S. ENGINEERS SURVEY-1941
SCALE: 1 IN. = 5000 FEET
1941

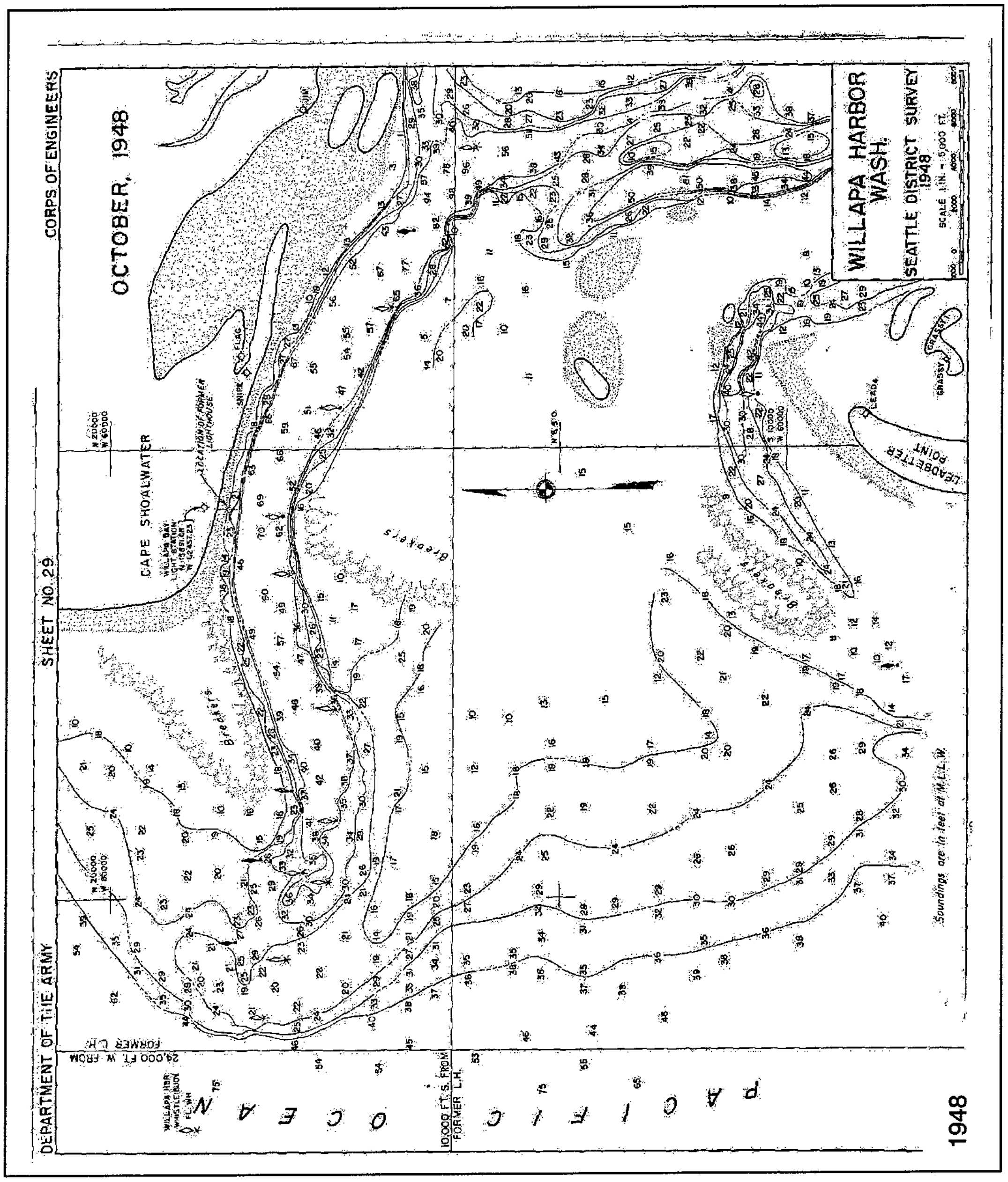

WAR DEPARTMENT
SHEET NO. 24
CORPS OF ENGINEERS, U.S. ARMY
MAY 1942
CAPE SHOALWATER
LIGHTHOUSE
LOCATION OF FORMER LIGHTHOUSE
Breakers
LEADBETTER PT.
24,000 FT. W. FROM FORMER L.H.
10,000 FT. S. FROM FORMER L.H.
PACIFIC OCEAN
WILLAPA HARBOR
WASH.
U.S. ENGINEERS SURVEY-1942
SCALE: 1 IN.=5,000 FEET
1942

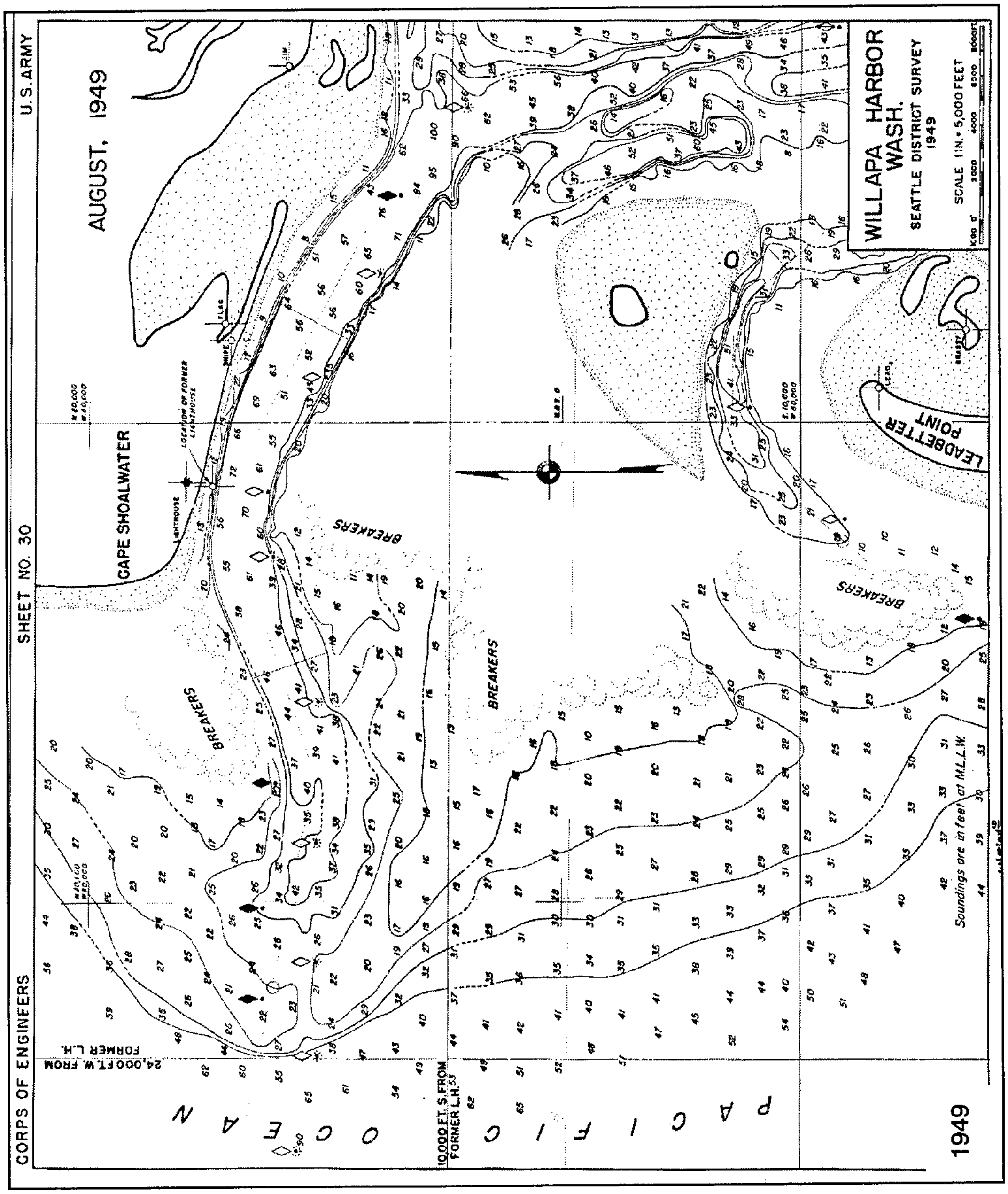

WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
SHEET NO. 25
JULY 1943
CAPE SHOALWATER
LIGHTHOUSE
LEADBETTER PT.
Breakers
PACIFIC
OCEAN
WILLAPA HARBOR
WASH.
U.S. ENGINEERS SURVEY 1943
SCALE: 1 IN. = 5,000 FEET
10,000 FT. S. FROM FORMER L.H.
24,000 FT. W. FROM FORMER L.H.
1943

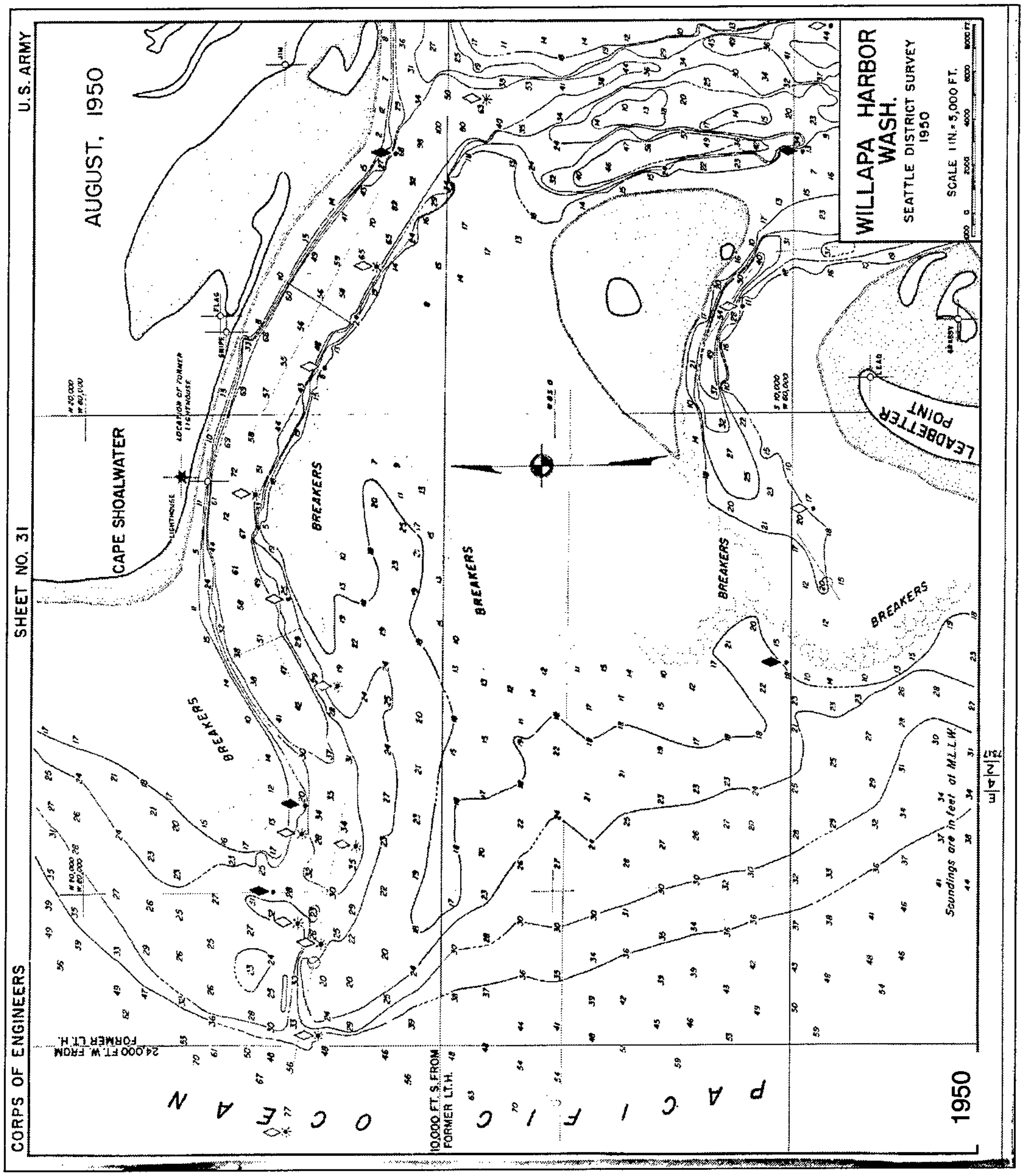

WAR DEPARTMENT
SHEET NO. 27
CORPS OF ENGINEERS U.S. ARMY
AUGUST, 1945
CAPE SHOALWATER
LIGHTHOUSE
LOCATION OF FORMER LIGHTHOUSE
Breakers
WILLAPA HARBOR
WASH.
U.S. ENGINEERS SURVEY 1945
SCALE 1 IN. = 5,000 FEET
LEADBETTER POINT
PACIFIC OCEAN
24,000 FT. W. FROM FORMER L.H.
10,000 FT. S. FROM FORMER L.H.
Soundings are in feet at M.L.L.W.
1945

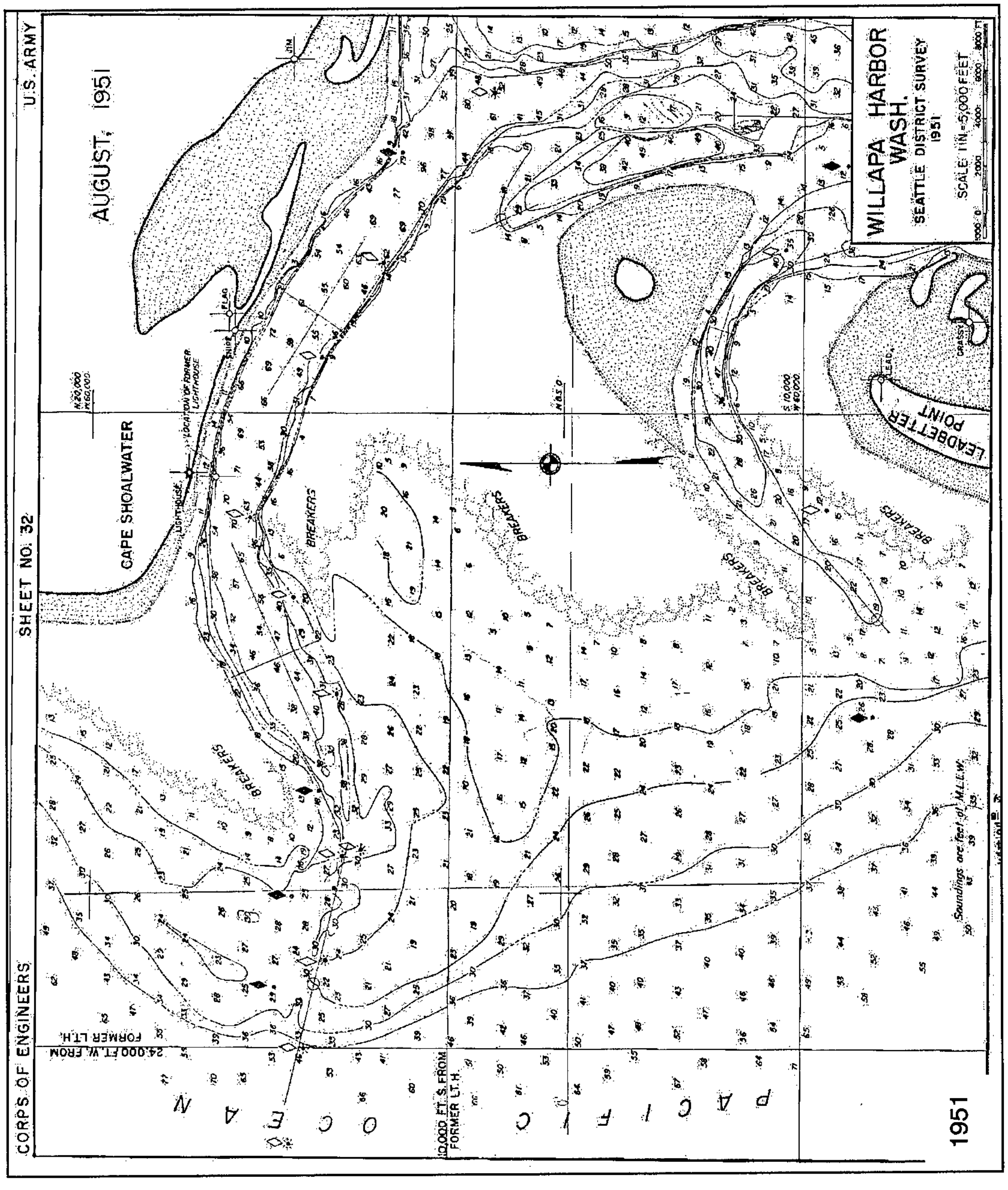


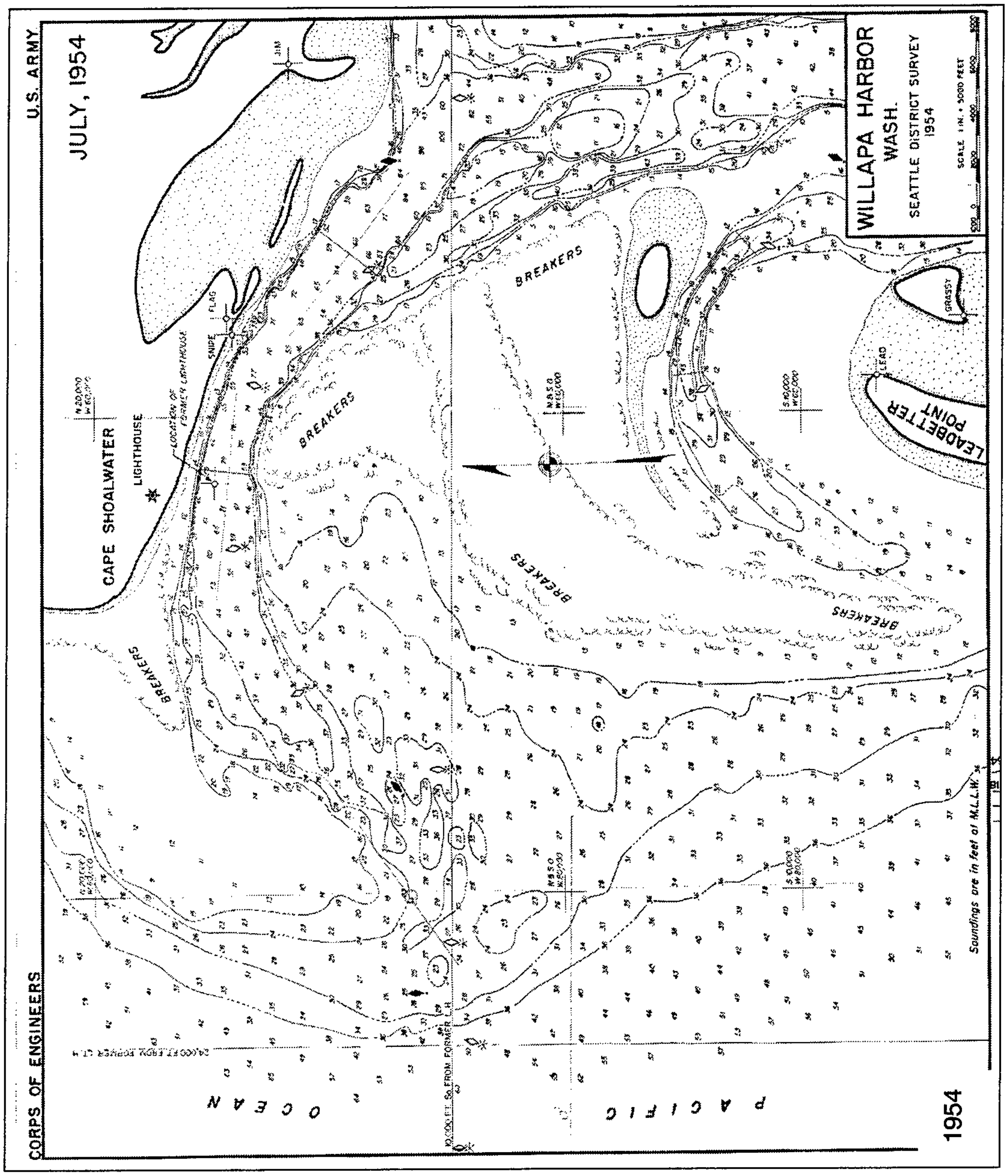
WAR DEPARTMENT
SHEET NO. 28
CORPS OF ENGINEERS U.S. ARMY
SEPTEMBER, 1946
CAPE SHOALWATER
LIGHTHOUSE
LOCATION OF FORMER LIGHTHOUSE
Breakers
PACIFIC OCEAN
LEADBETTER POINT
10,000 FT. S. FROM FORMER L.H.
24,000 FT. W. FROM FORMER L.H.
Soundings are in feet at M.L.L.W.
WILLAPA HARBOR WASH.
U.S. ENGINEERS SURVEY 1946
SCALE 1 IN. = 5,000 FEET
1946

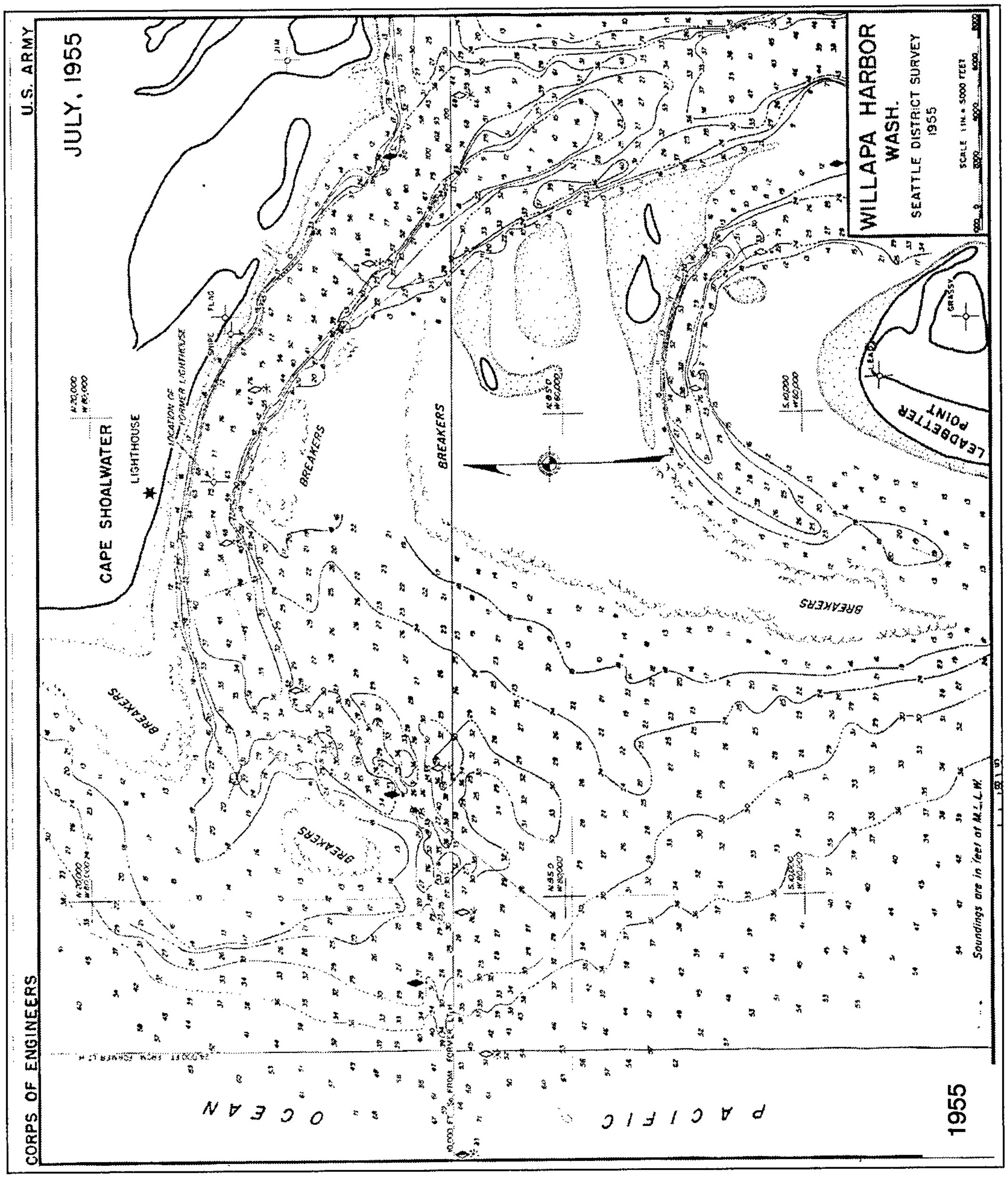

DEPARTMENT OF THE ARMY
SHEET NO. 29
CORPS OF ENGINEERS
OCTOBER, 1948
CAPE SHOALWATER
LEADBETTER POINT
PACIFIC OCEAN
Soundings are in feet at M.L.L.W.
WILLAPA HARBOR WASH.
SEATTLE DISTRICT SURVEY 1948
SCALE 1 IN. = 5,000 FT.
1948

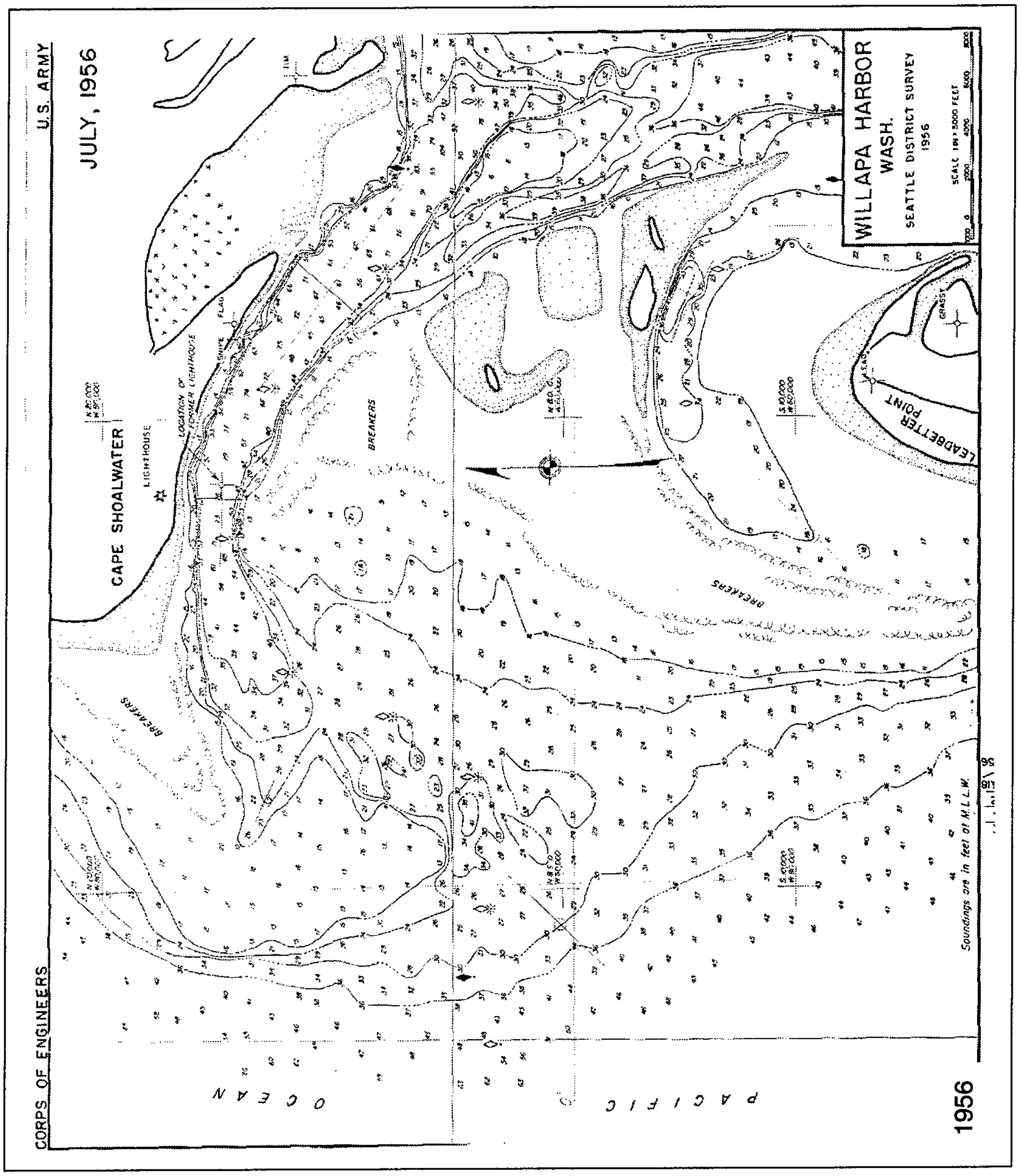

CORPS OF ENGINEERS
SHEET NO. 30
U.S. ARMY
AUGUST, 1949
CAPE SHOALWATER
LIGHTHOUSE
LOCATION OF FORMER LIGHTHOUSE
BREAKERS
LEADBETTER POINT
OCEAN
PACIFIC
24,000 FT. W. FROM FORMER L.H.
10,000 FT. S. FROM FORMER L.H.
Soundings are in feet at M.L.L.W.
WILLAPA HARBOR WASH.
SEATTLE DISTRICT SURVEY
1949
SCALE 1 IN. = 5,000 FEET
1949

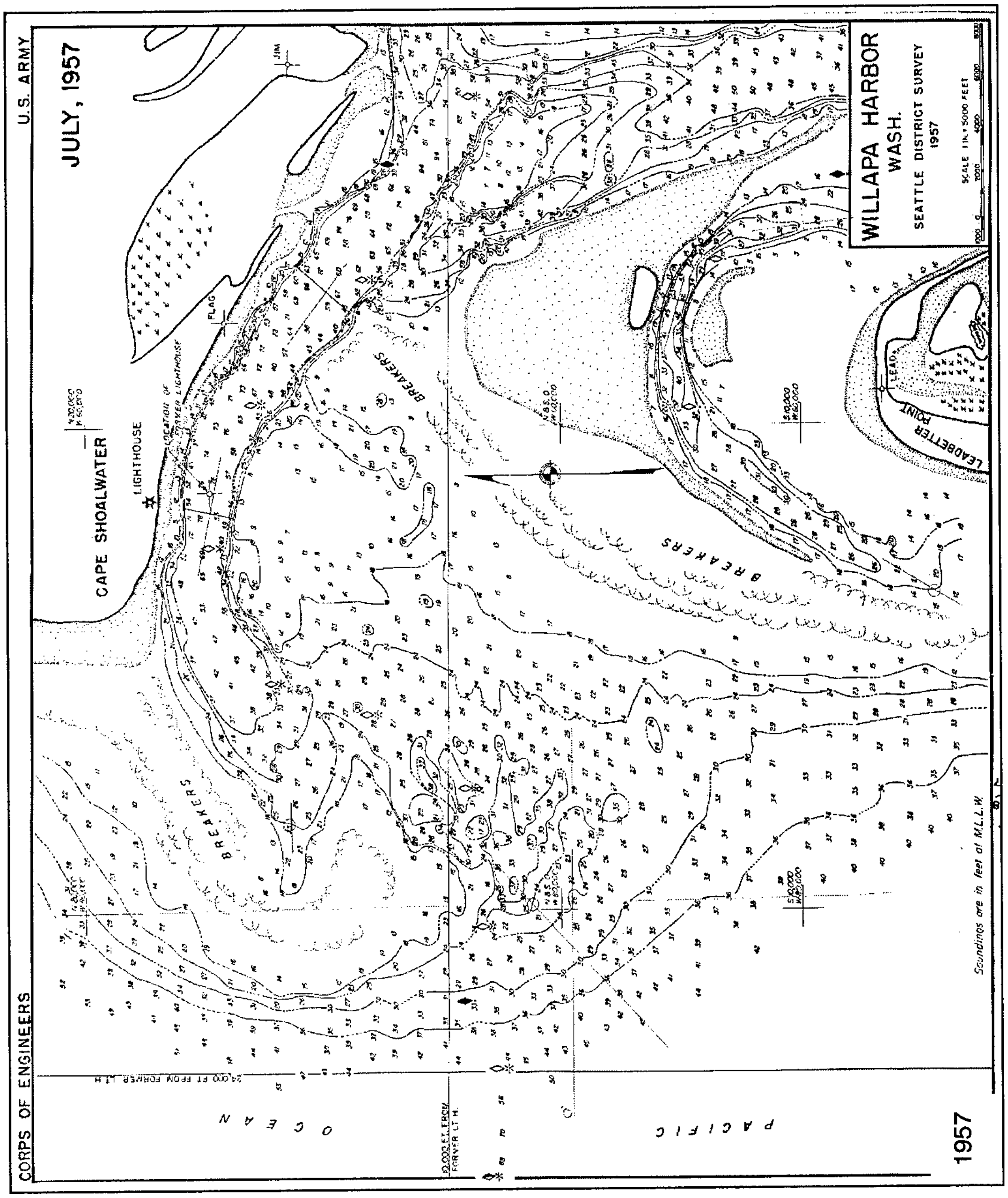

CORPS OF ENGINEERS
SHEET NO. 31
U.S. ARMY
AUGUST, 1950
CAPE SHOALWATER
LEADBETTER POINT
BREAKERS
BREAKERS
BREAKERS
BREAKERS
BREAKERS
PACIFIC OCEAN
WILLAPA HARBOR WASH.
SEATTLE DISTRICT SURVEY 1950
SCALE 1 IN.=5,000 FT.
Soundings are in feet at M.L.L.W.
24,000 FT. W. FROM FORMER LT. H.
10,000 FT. S. FROM FORMER LT. H.
1950


CORPS OF ENGINEERS
SHEET NO. 32
U.S. ARMY
AUGUST, 1951
CAPE SHOALWATER
LEADBETTER POINT
BREAKERS
PACIFIC OCEAN
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1951
SCALE 1 IN.=5,000 FEET
Soundings are feet at M.L.L.W.
24,000 FT. W. FROM FORMER LT.H.
10,000 FT. S. FROM FORMER LT.H.
1951

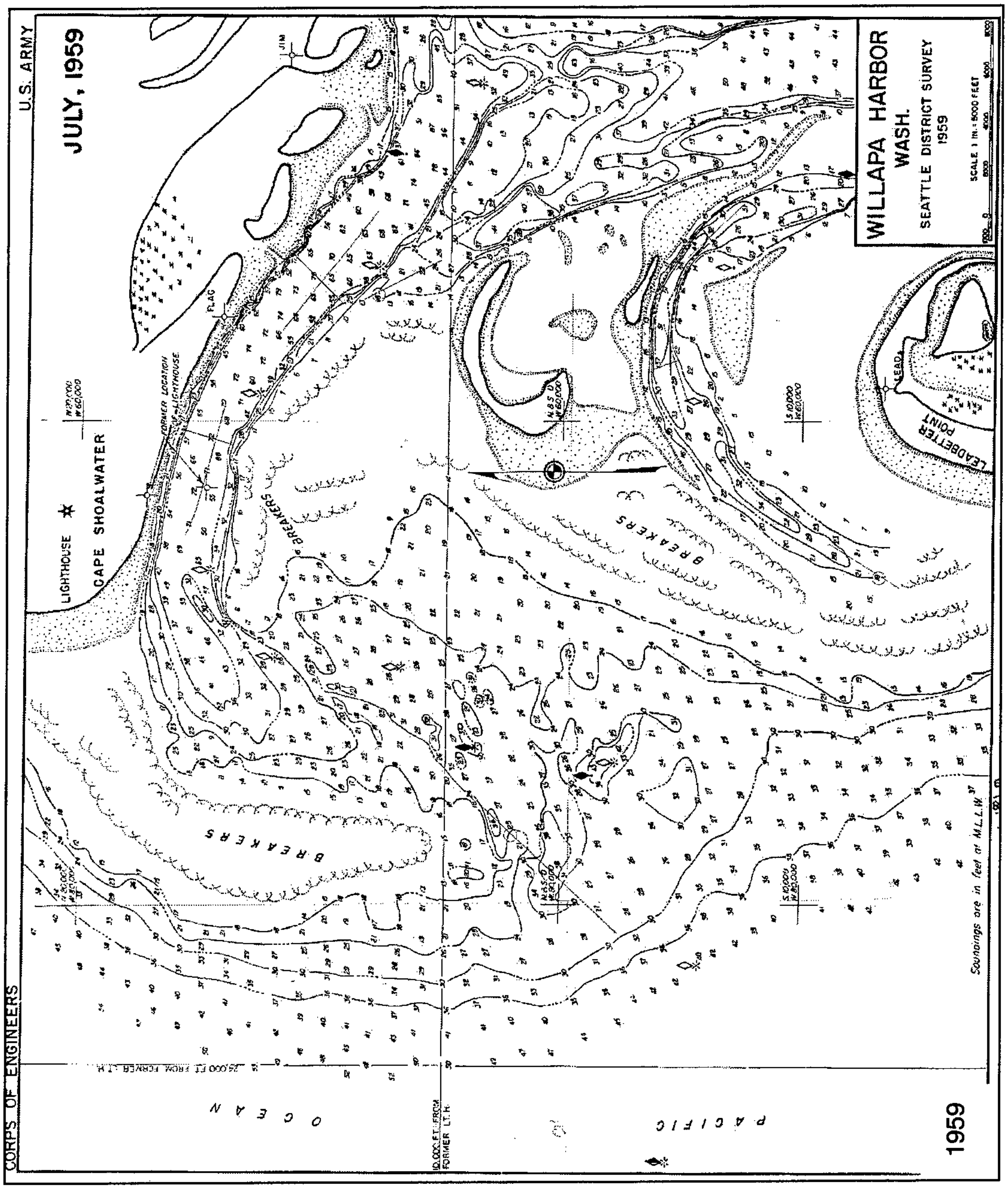

U.S. ARMY
CORPS OF ENGINEERS
JULY, 1954
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1954
CAPE SHOALWATER
LIGHTHOUSE
BREAKERS
LEADBETTER POINT
GRASSY
PACIFIC OCEAN
Soundings are in feet at M.L.L.W.
1954


CORPS OF ENGINEERS
U.S. ARMY
JULY, 1955
CAPE SHOALWATER
LIGHTHOUSE
LOCATION OF FORMER LIGHTHOUSE
BREAKERS
BREAKERS
BREAKERS
BREAKERS
BREAKERS
LEADBETTER POINT
GRASSY
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1955
PACIFIC
OCEAN
Soundings are in feet at M.L.L.W.
1955

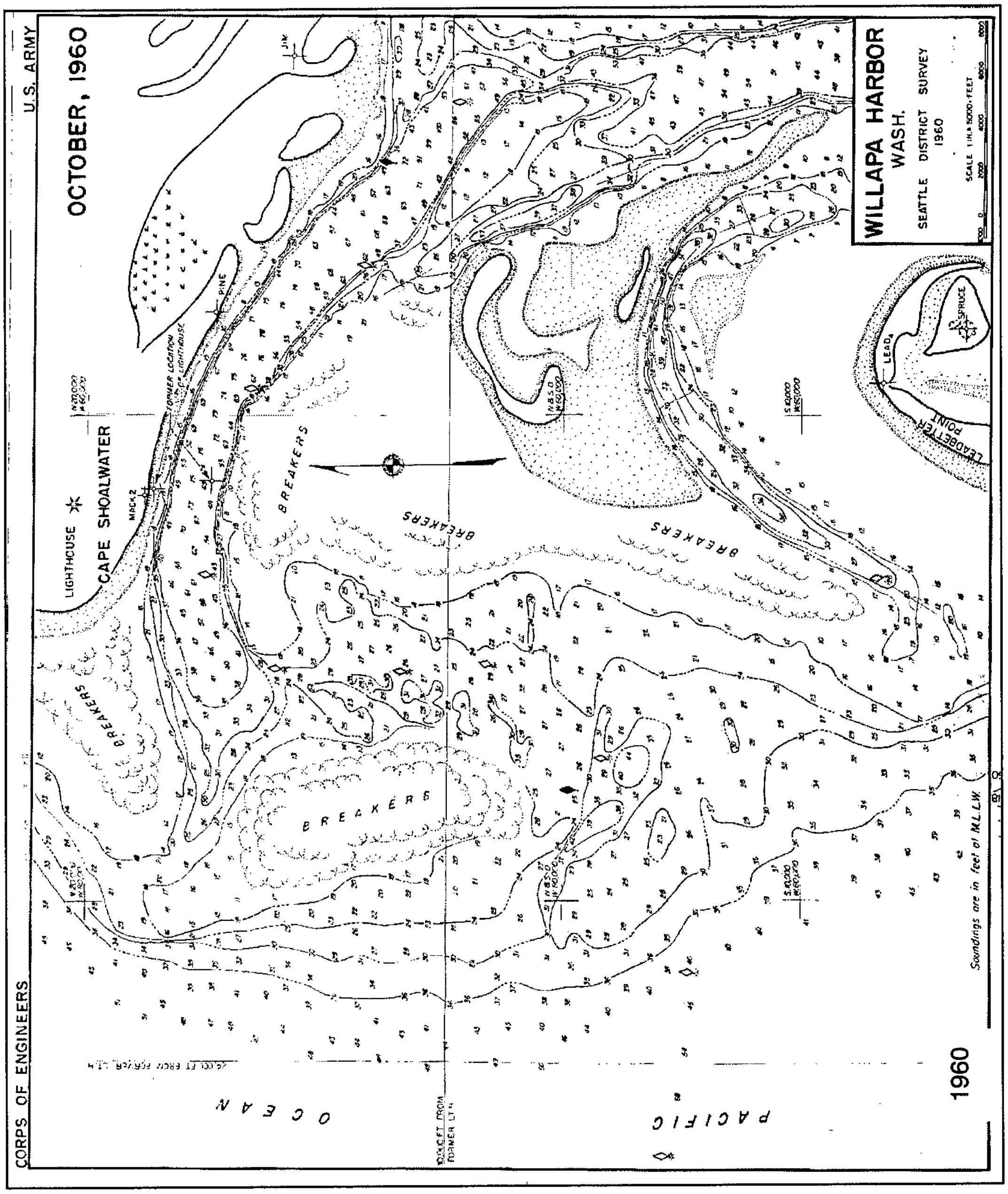

CORPS OF ENGINEERS
U.S. ARMY
JULY, 1956
CAPE SHOALWATER
LIGHTHOUSE
LOCATION OF FORMER LIGHTHOUSE
BREAKERS
LEADBETTER POINT
GRASSY
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1956
PACIFIC OCEAN
Soundings are in feet of M.L.L.W.
1956

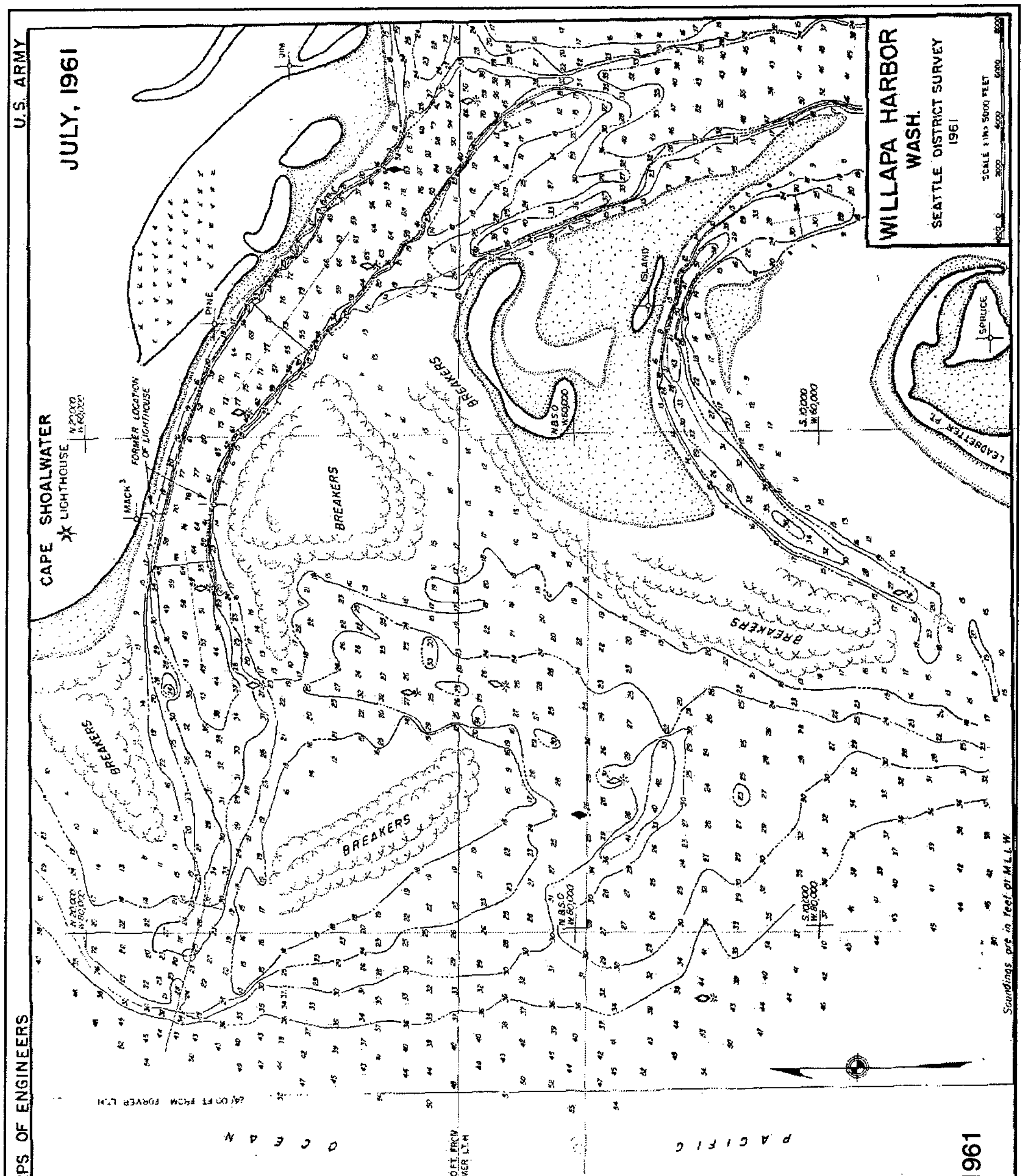


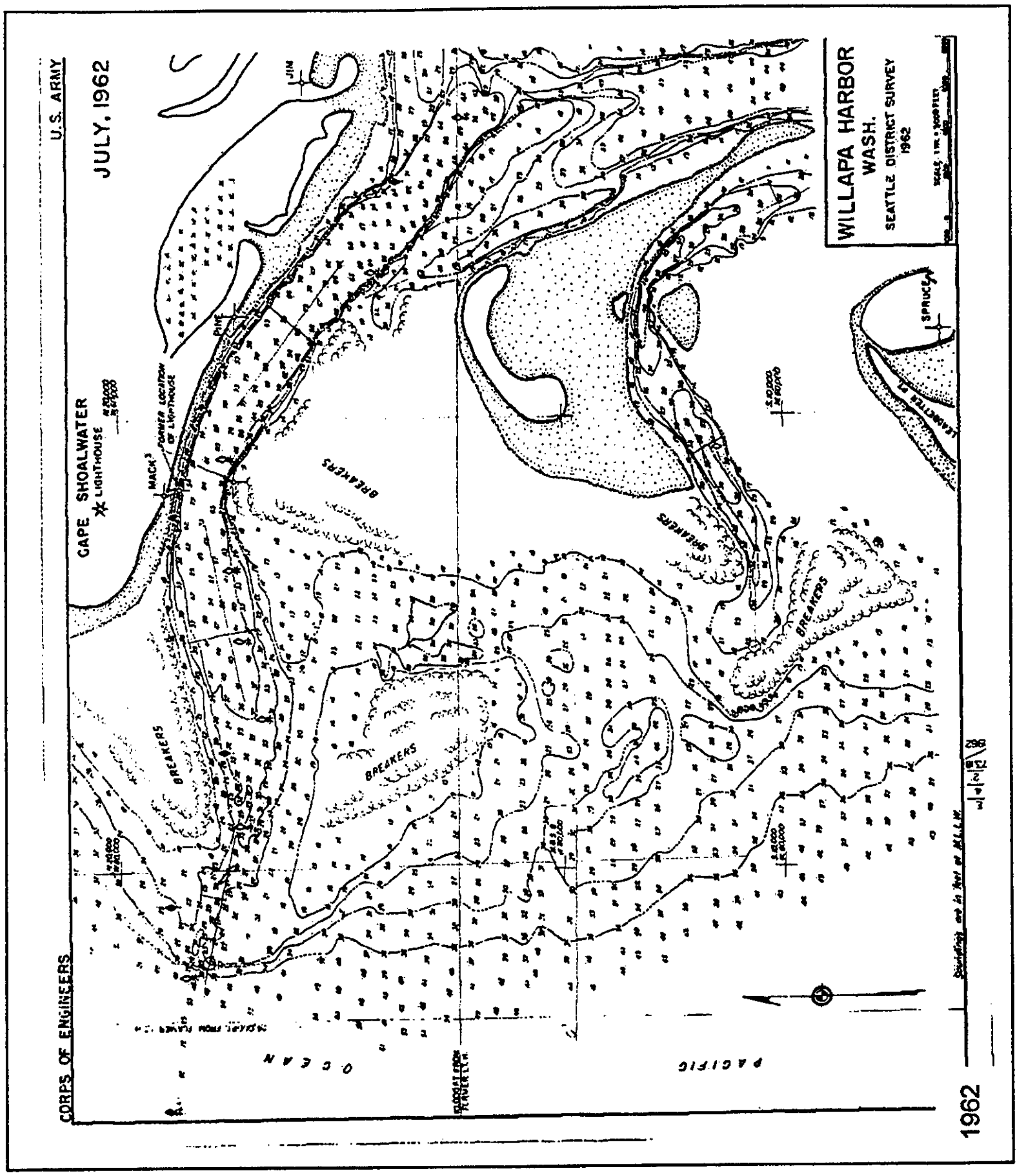
U.S. ARMY
CORPS OF ENGINEERS
JULY, 1957
CAPE SHOALWATER
LIGHTHOUSE
LOCATION OF FORMER LIGHTHOUSE
FLAG
BREAKERS
BREAKERS
BREAKERS
LEADBETTER POINT
LEAD
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1957
OCEAN
PACIFIC
Soundings are in feet at M.L.L.W.
1957

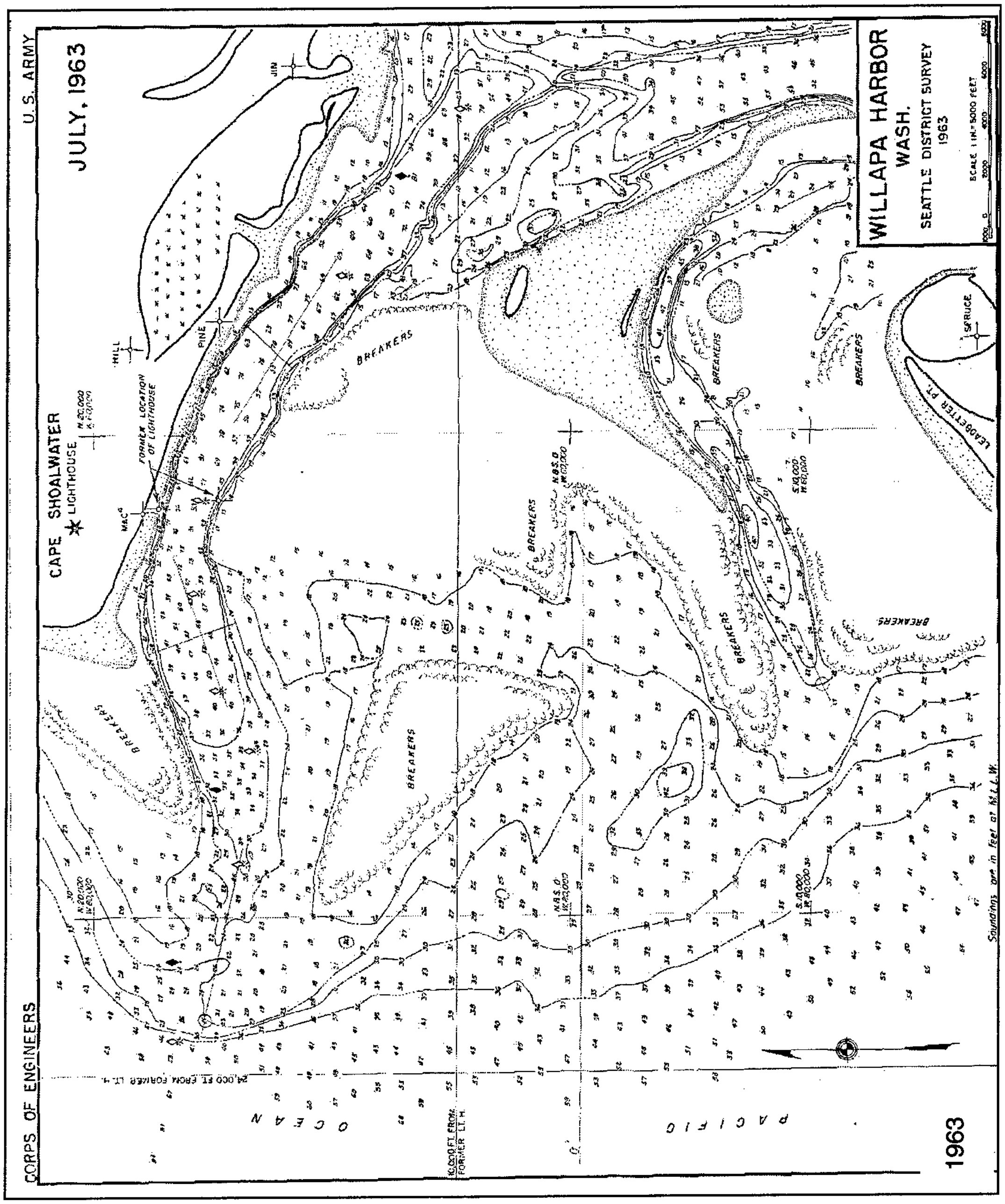


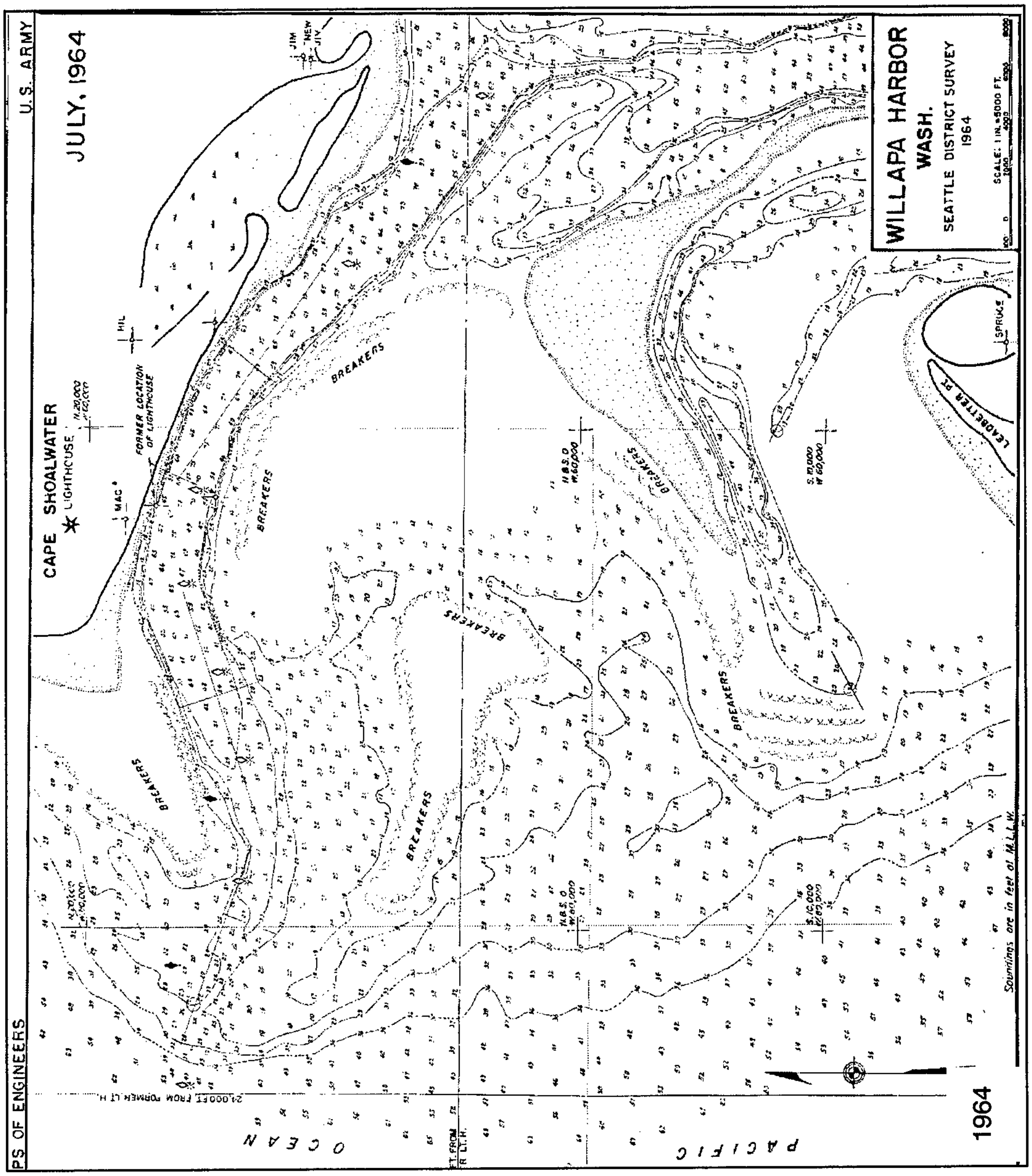
U.S. ARMY
CORPS OF ENGINEERS
JULY, 1959
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1959
LIGHTHOUSE
CAPE SHOALWATER
FORMER LOCATION
LIGHTHOUSE
FLAG
LEADBETTER POINT
BREAKERS
BREAKERS
BREAKERS
O C E A N
P A C I F I C
10,000 FT. FROM FORMER L.H.
Soundings are in feet at M.L.L.W.
1959

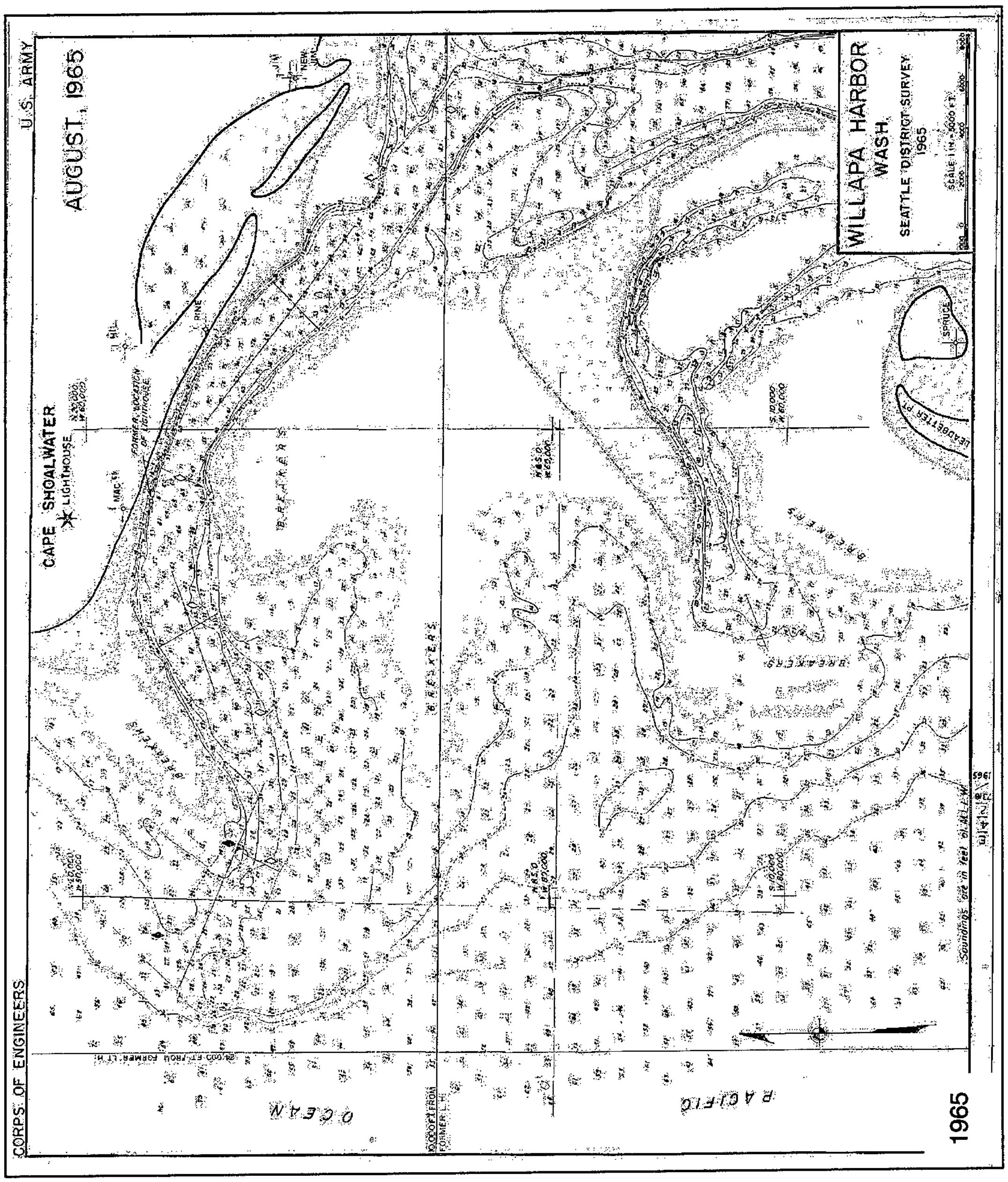

U.S. ARMY
CORPS OF ENGINEERS
OCTOBER, 1960
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1960
LIGHTHOUSE
CAPE SHOALWATER
BREAKERS
BREAKERS
BREAKERS
BREAKERS
BREAKERS
LEADBETTER POINT
OCEAN
PACIFIC
Soundings are in feet at M.L.L.W.
1960

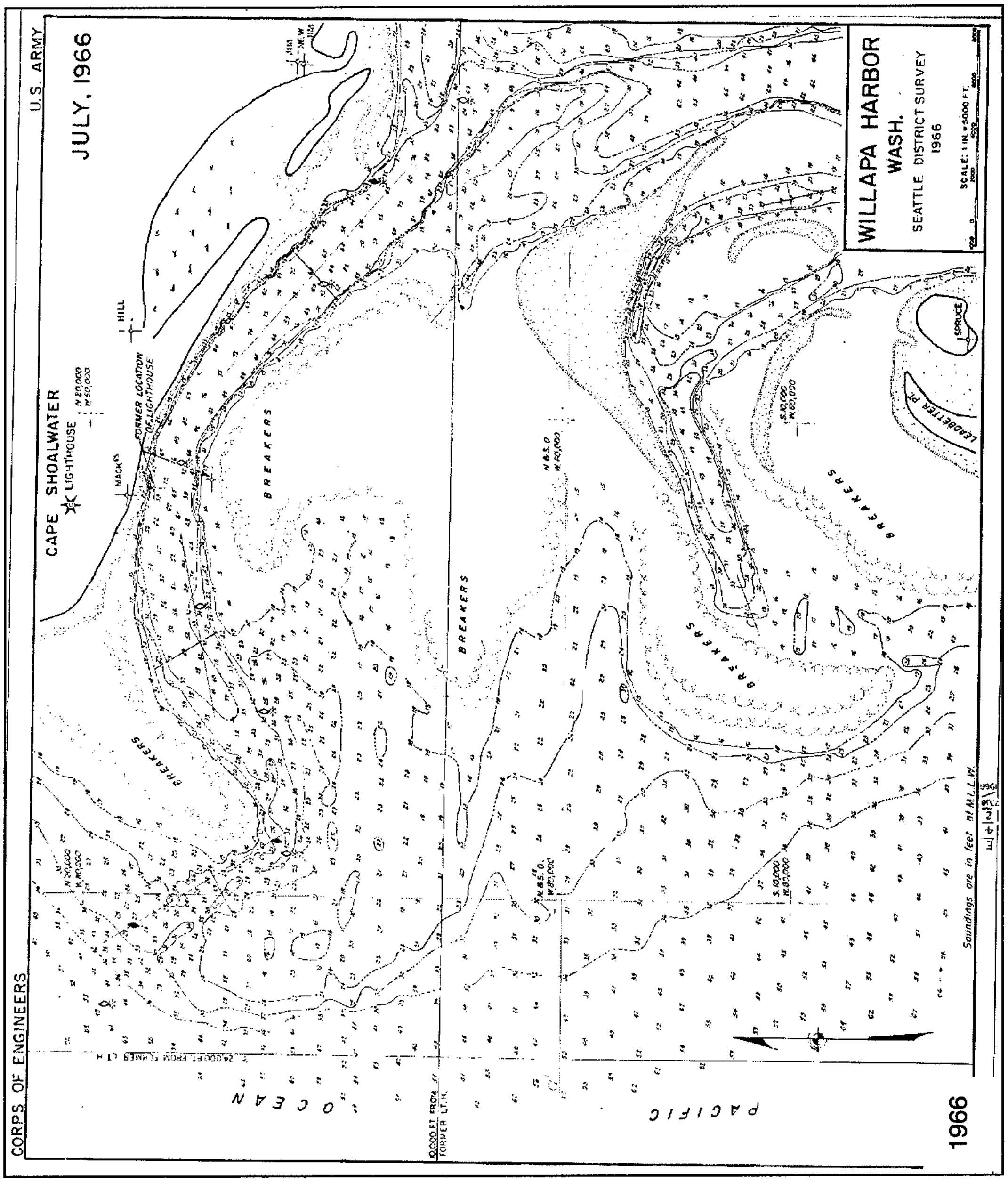

U.S. ARMY
CORPS OF ENGINEERS
JULY, 1961
CAPE SHOALWATER
LIGHTHOUSE
FORMER LOCATION OF LIGHTHOUSE
BREAKERS
BREAKERS
BREAKERS
BREAKERS
BREAKERS
ISLAND
LEADBETTER PT
SPRUCE
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1961
OCEAN
PACIFIC
10,000 FT. FROM FORMER LT.H.
Soundings are in feet at M.L.L.W.
1961

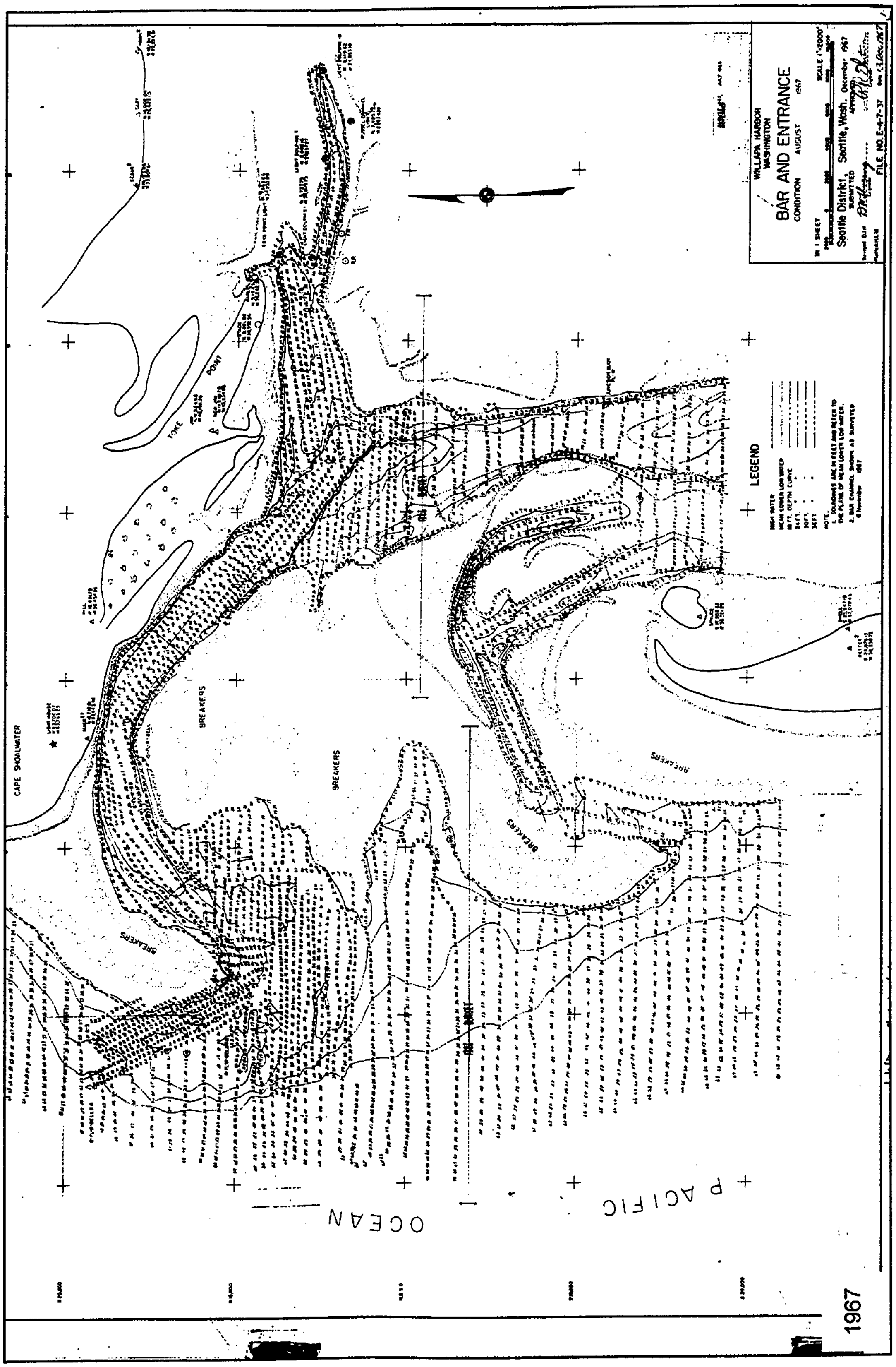

U.S. ARMY
JULY, 1962
CAPE SHOALWATER
LIGHTHOUSE
FORMER LOCATION OF LIGHTHOUSE
BREAKERS
BREAKERS
BREAKERS
BREAKERS
BREAKERS
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1962
SPRUCE
PACIFIC
OCEAN
CORPS OF ENGINEERS
1962

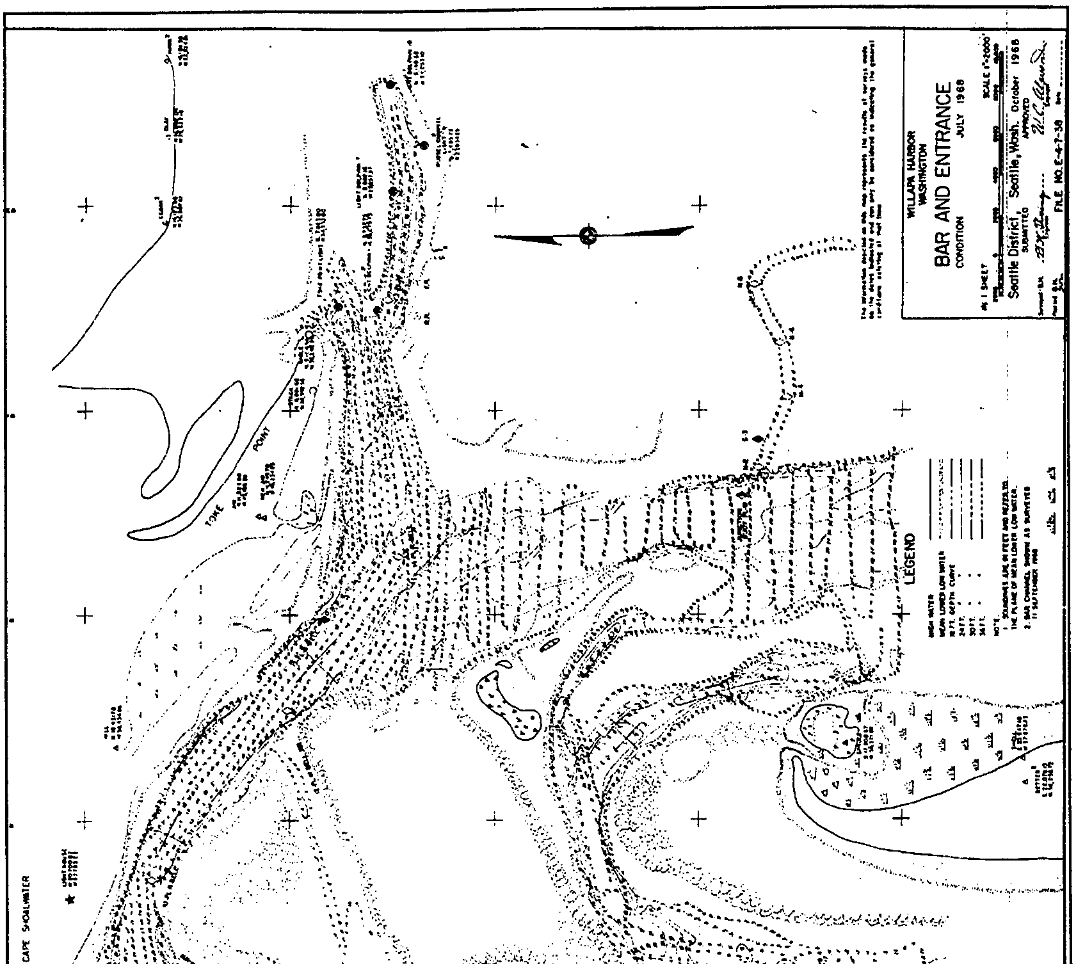

CORPS OF ENGINEERS
U.S. ARMY
JULY, 1963
CAPE SHOALWATER
LIGHTHOUSE
FORMER LOCATION OF LIGHTHOUSE
BREAKERS
LEADBETTER PT.
SPRUCE
PACIFIC OCEAN
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1963
Soundings are in feet at M.L.L.W.
1963

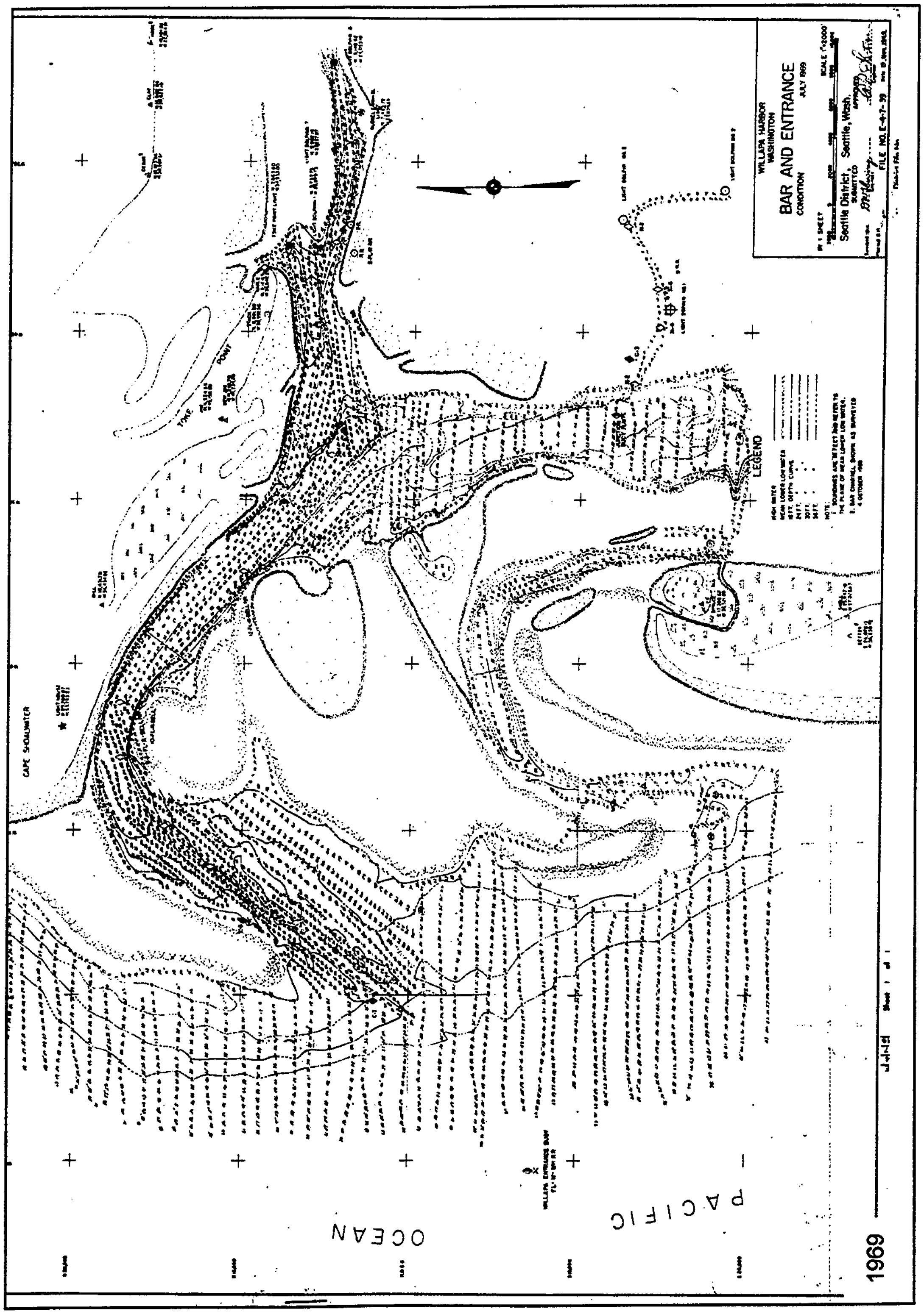

PS OF ENGINEERS
U.S. ARMY
JULY, 1964
CAPE SHOALWATER
LIGHTHOUSE
FORMER LOCATION OF LIGHTHOUSE
BREAKERS
LEADBETTER PT.
SPRUCE
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1964
Soundings are in feet at M.L.L.W.
OCEAN
PACIFIC
1964

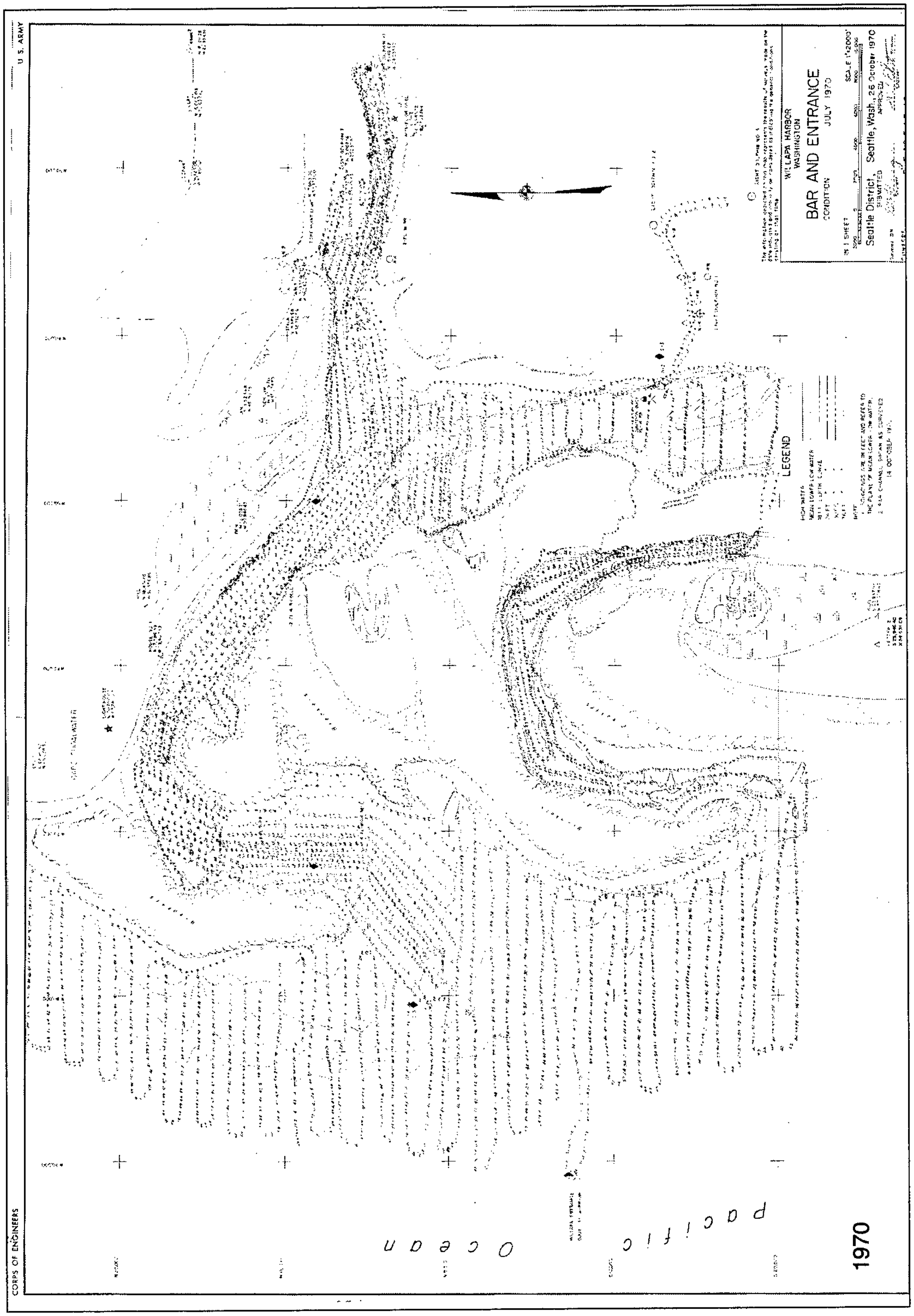

U.S. ARMY
CORPS OF ENGINEERS
AUGUST, 1965
CAPE SHOALWATER
LIGHTHOUSE
FORMER LOCATION OF LIGHTHOUSE
BREAKERS
OCEAN
PACIFIC
SPRUCE
LEADBETTER PT.
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1965
Soundings are in feet at M.L.L.W.
1965

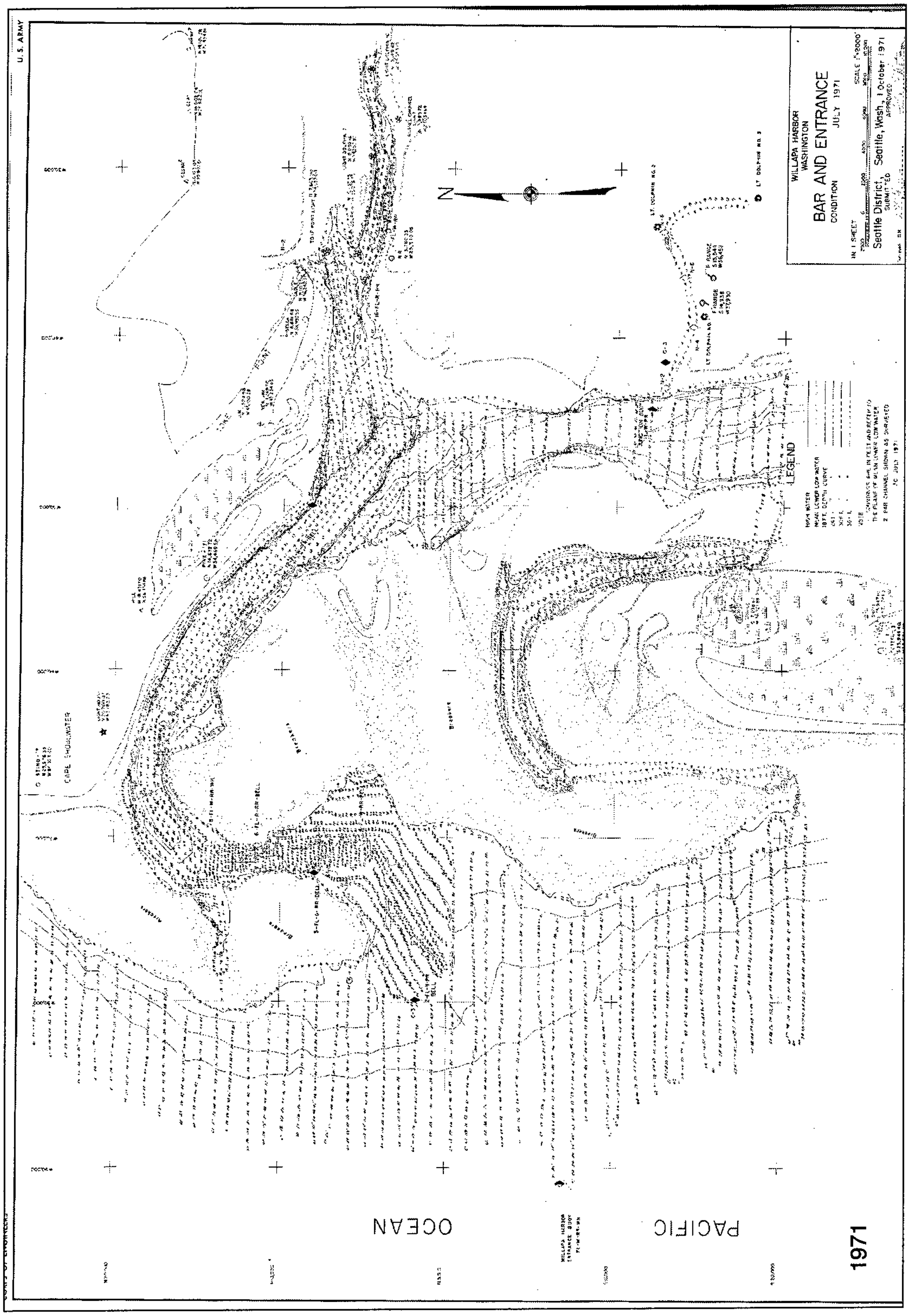

U.S. ARMY
CORPS OF ENGINEERS
JULY, 1966
CAPE SHOALWATER
LIGHTHOUSE
FORMER LOCATION OF LIGHTHOUSE
HILL
BREAKERS
BREAKERS
BREAKERS
BREAKERS
LEADBETTER PT.
SPRUCE
OCEAN
PACIFIC
WILLAPA HARBOR
WASH.
SEATTLE DISTRICT SURVEY
1966
SCALE: 1 IN. = 5000 FT.
Soundings are in feet at M.L.L.W.
1966

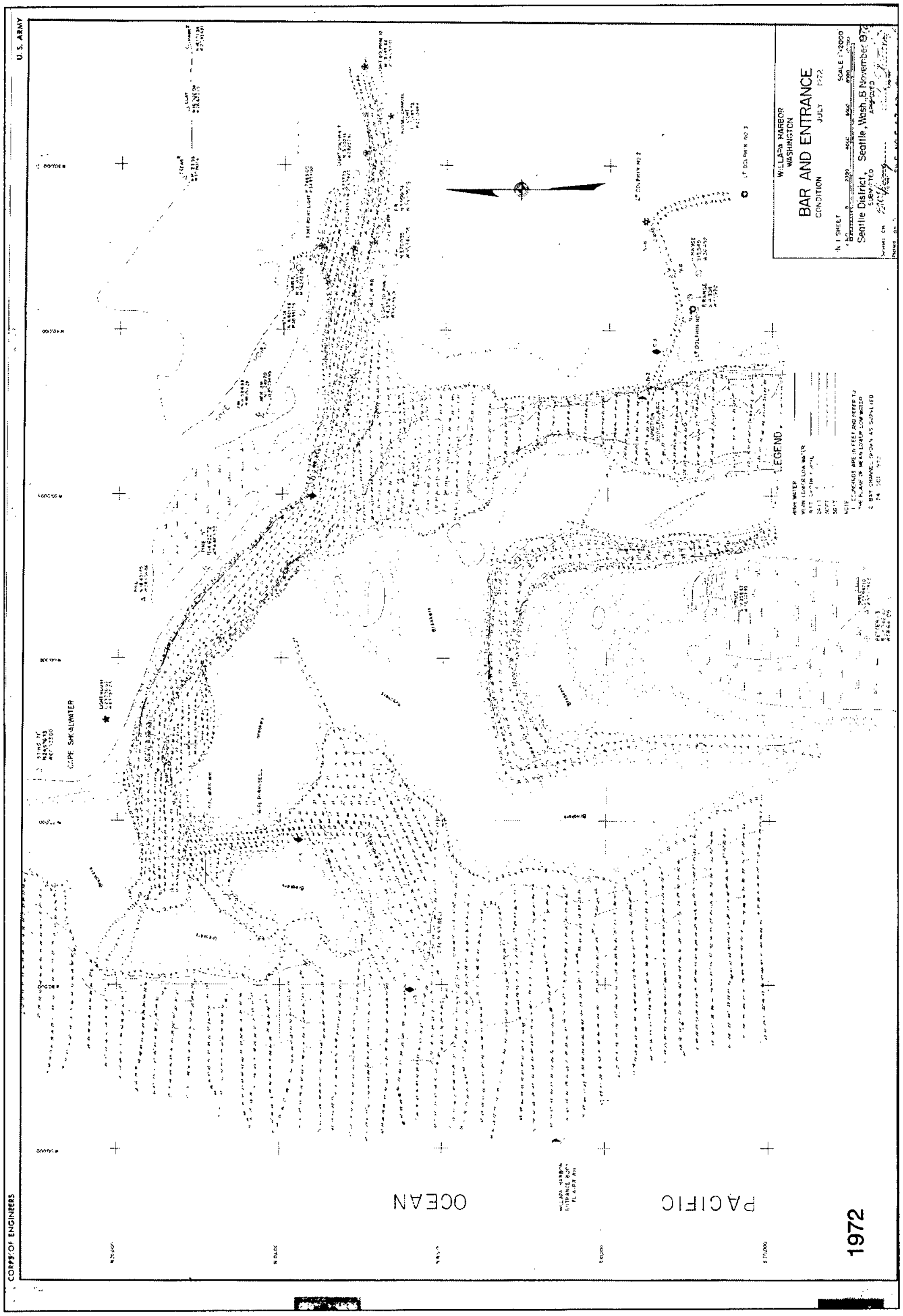

WILLAPA HARBOR
WASHINGTON
BAR AND ENTRANCE
CONDITION AUGUST 1967
Seattle District, Seattle, Wash. December 1967
LEGEND
CAPE SHOALWATER
TOKE POINT
BREAKERS
PACIFIC OCEAN
1967

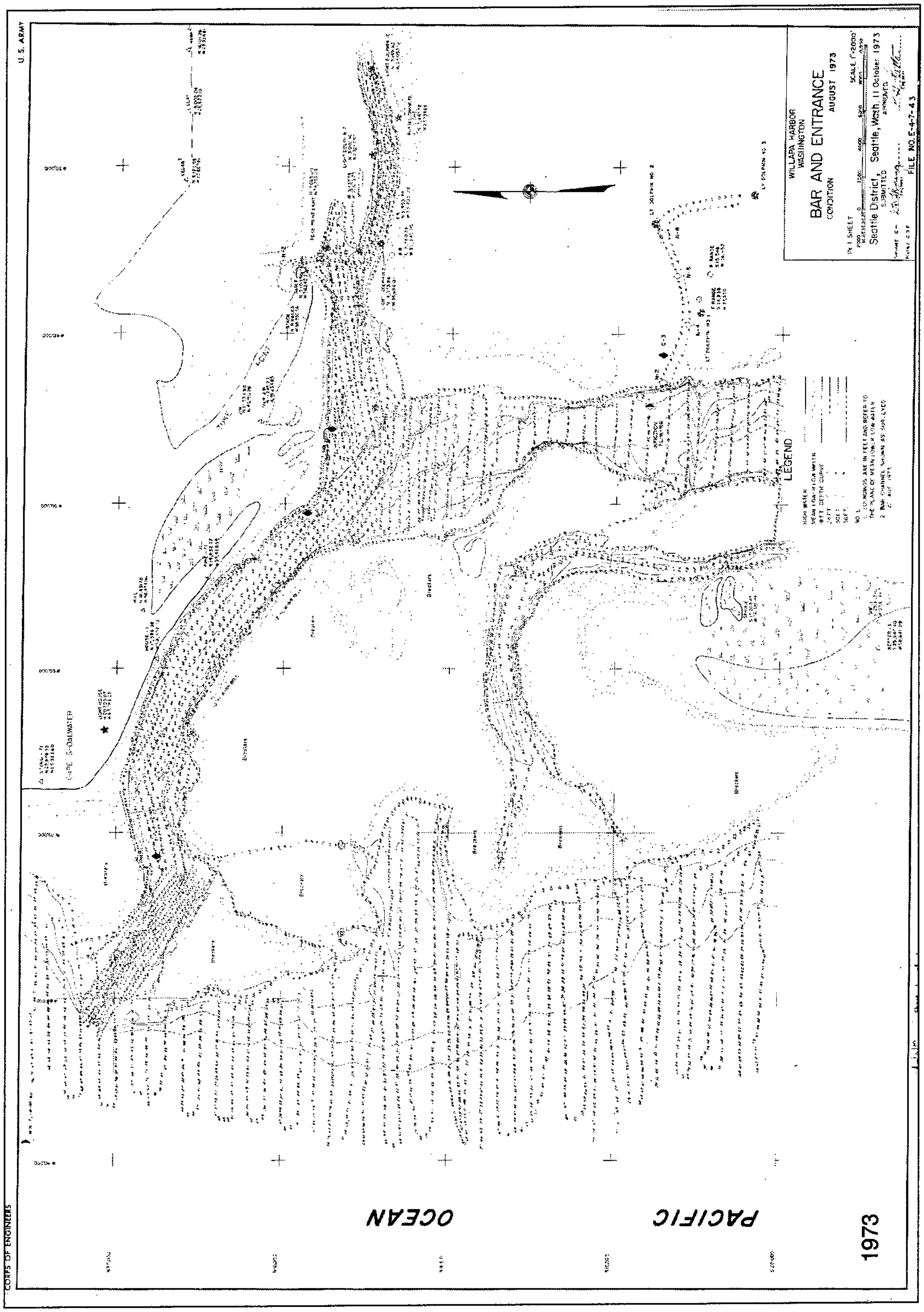

WILLAPA HARBOR
WASHINGTON
BAR AND ENTRANCE
CONDITION
JULY 1968
Seattle District, Seattle, Wash. October 1968
LEGEND
CAPE SHOALWATER
TOKE POINT
1968

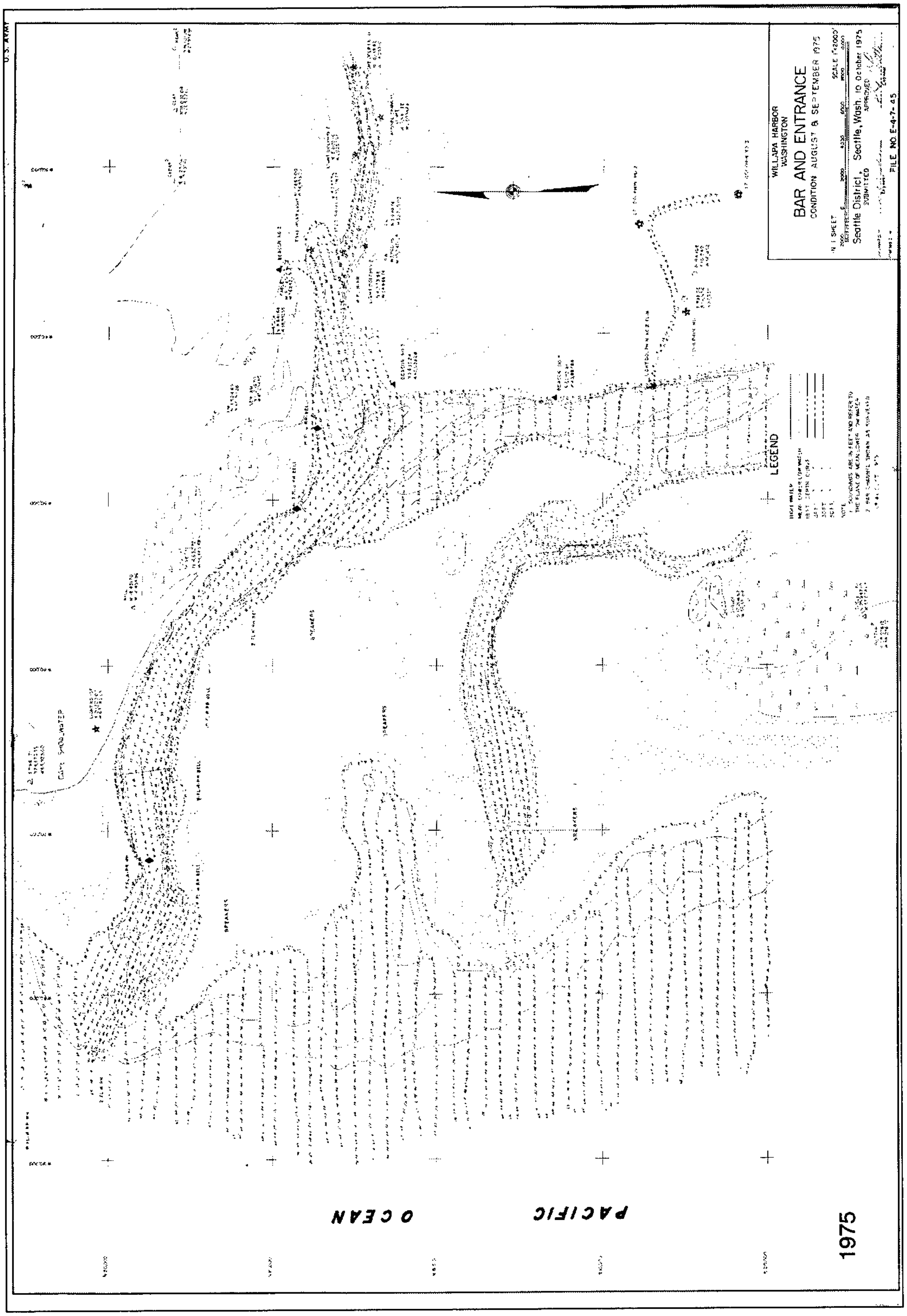

WILLAPA HARBOR
WASHINGTON
BAR AND ENTRANCE
CONDITION
JULY 1969
Seattle District, Seattle, Wash.
LEGEND
CAPE SHOALWATER
PACIFIC
OCEAN
1969


U.S. ARMY
CORPS OF ENGINEERS
WILLAPA HARBOR
WASHINGTON
BAR AND ENTRANCE
CONDITION
JULY 1970
Seattle District, Seattle, Wash., 26 October 1970
LEGEND
Pacific
Ocean
1970

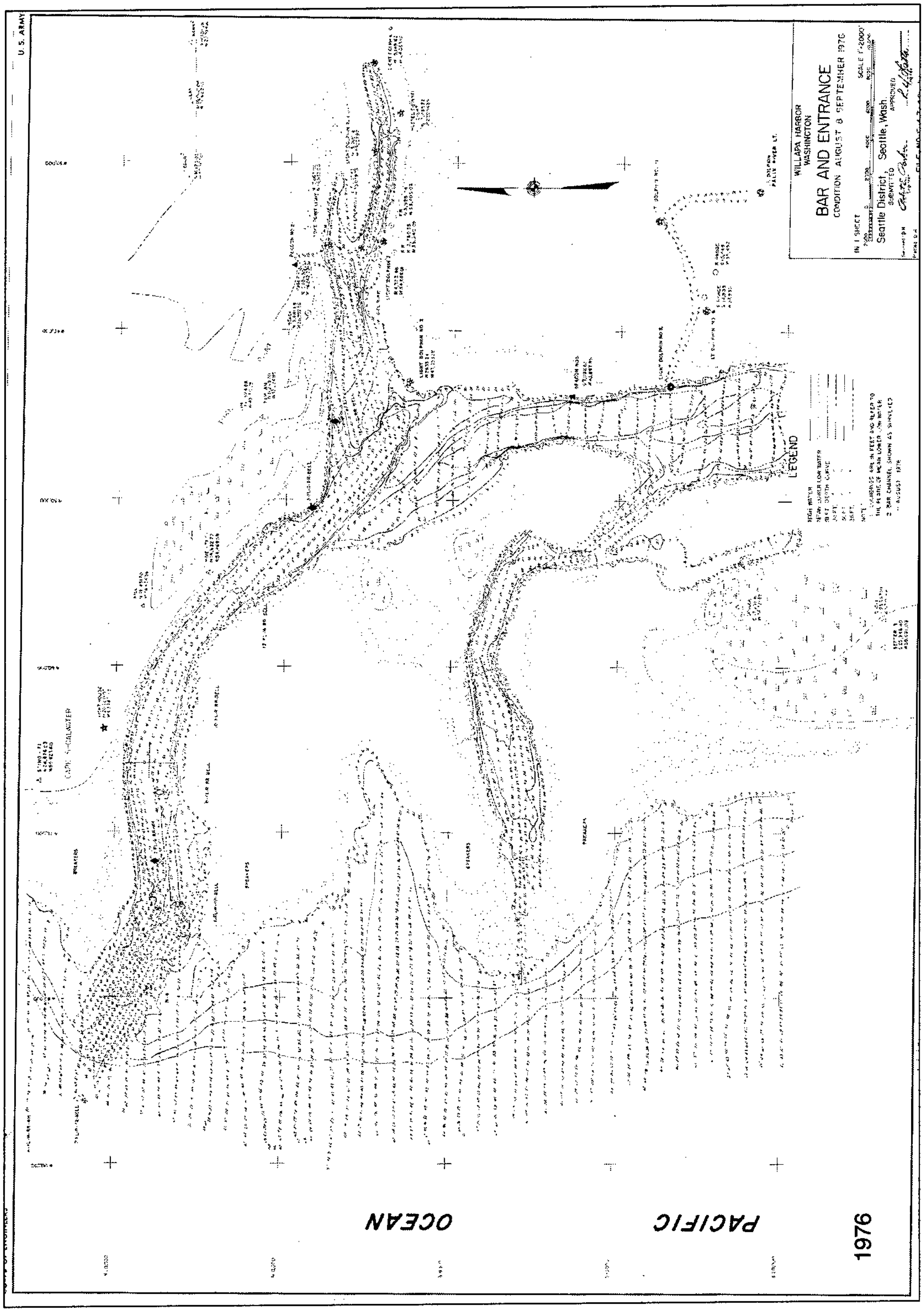

U.S. ARMY
WILLAPA HARBOR
WASHINGTON
BAR AND ENTRANCE
CONDITION JULY 1971
Seattle District, Seattle, Wash., 1 October 1971
LEGEND
CAPE SHOALWATER
PACIFIC OCEAN
1971

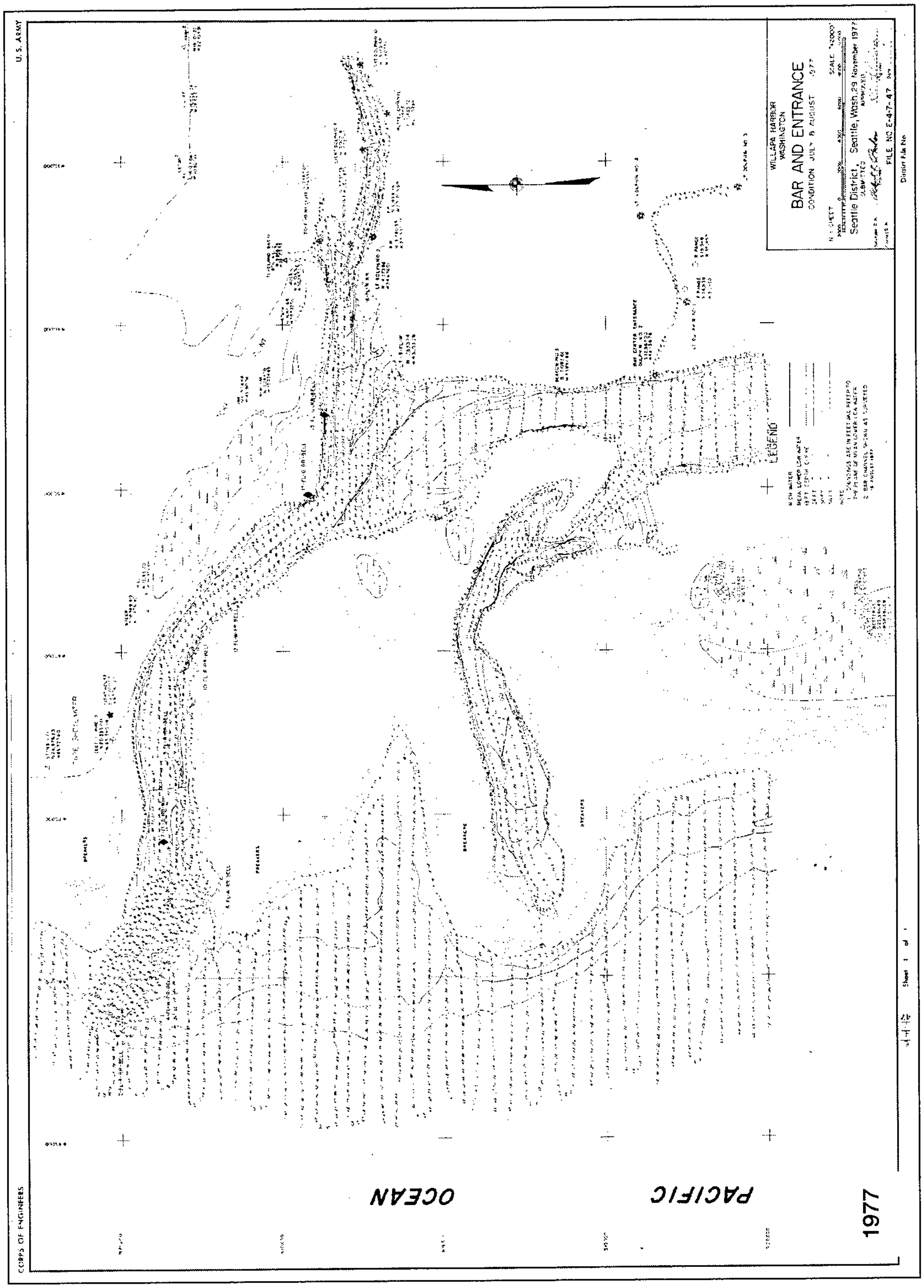

U.S. ARMY
CORPS OF ENGINEERS
WILLAPA HARBOR
WASHINGTON
BAR AND ENTRANCE
CONDITION JULY 1972
Seattle District, Seattle, Wash., 8 November 1972
CAPE SHOALWATER
LEGEND
PACIFIC
OCEAN
1972

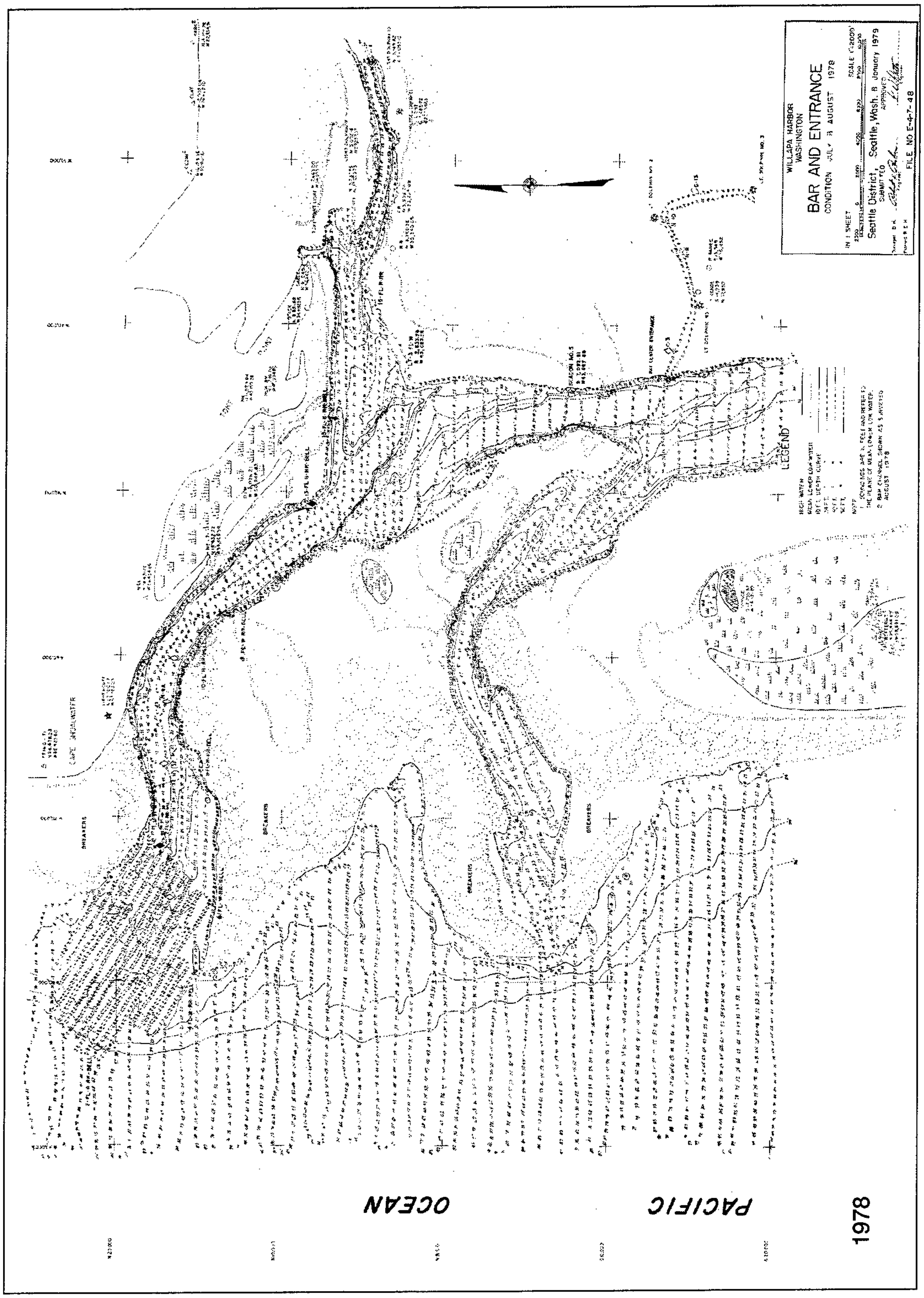

U.S. ARMY
CORPS OF ENGINEERS
WILLAPA HARBOR
WASHINGTON
BAR AND ENTRANCE
CONDITION
AUGUST 1973
Seattle District, Seattle, Wash. 11 October 1973
FILE NO. E-4-7-43
LEGEND
CAPE SHOALWATER
PACIFIC
OCEAN
1973

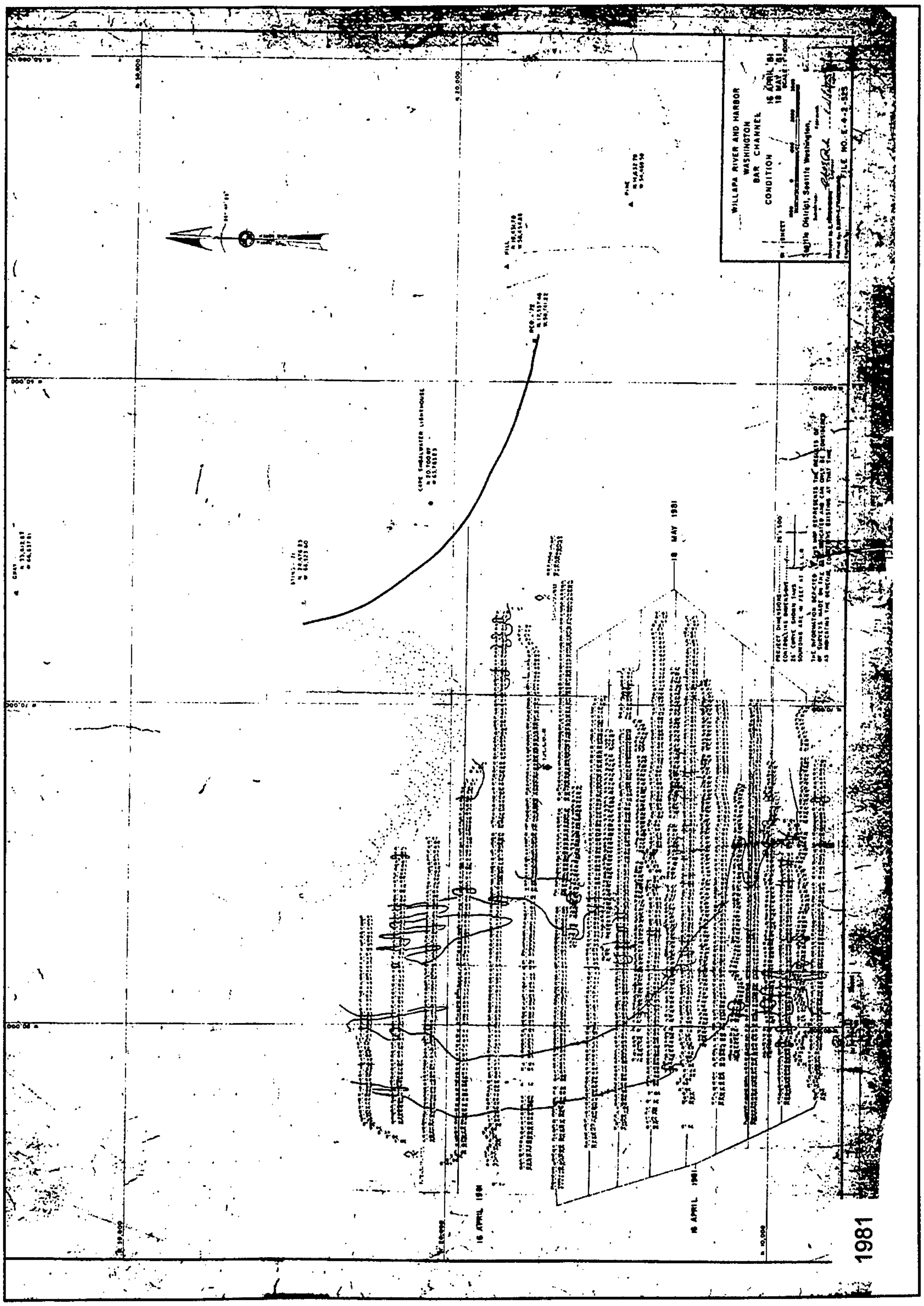


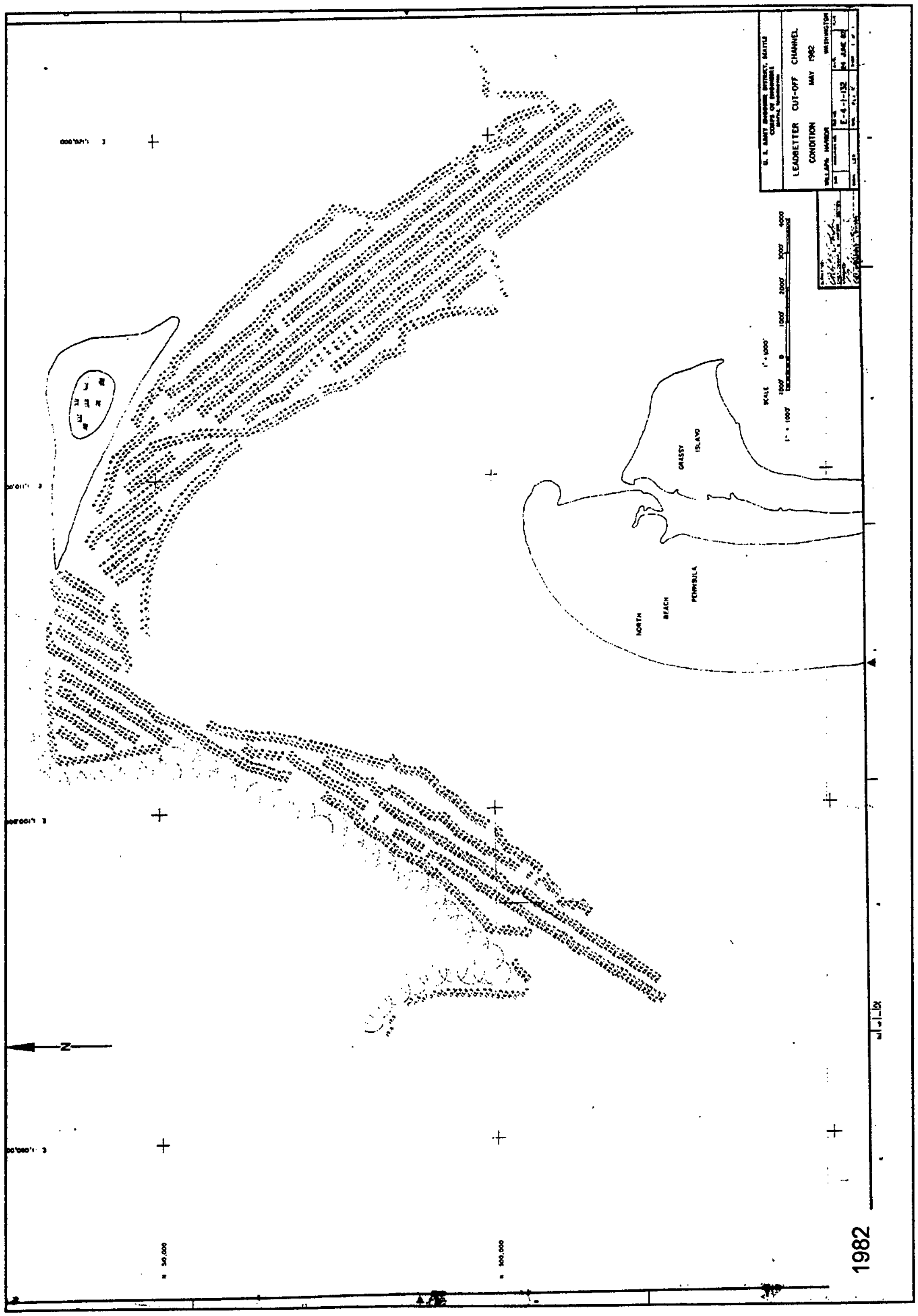
WILLAPA HARBOR
WASHINGTON
BAR AND ENTRANCE
Seattle District
FILE NO. E-4-7-45
LEGEND
PACIFIC
OCEAN
1975

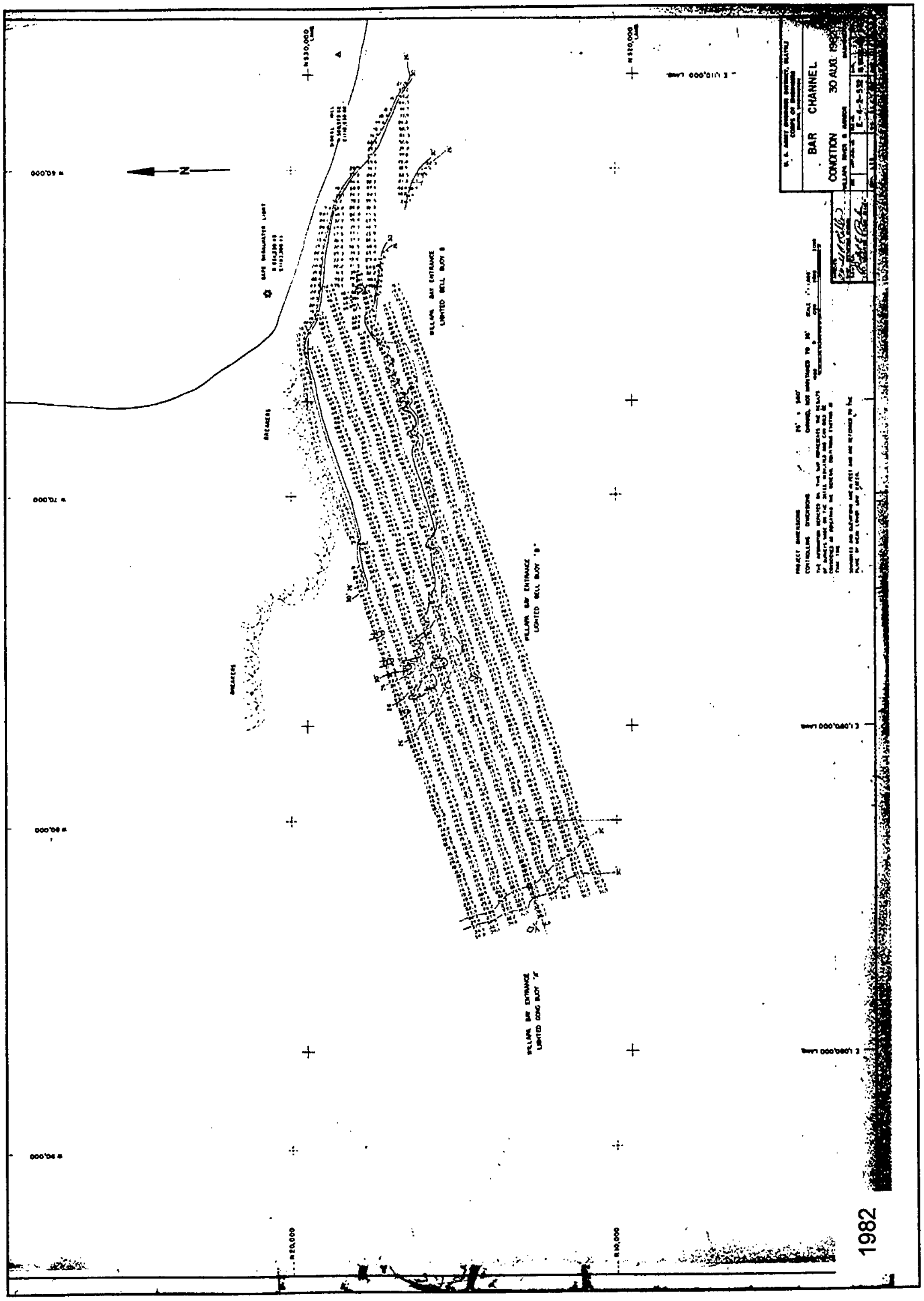

WILLAPA HARBOR
WASHINGTON
BAR AND ENTRANCE
CONDITION AUGUST 8 SEPTEMBER 1976
Seattle District, Seattle, Wash.
LEGEND
PACIFIC OCEAN
1976
U. S. ARMY

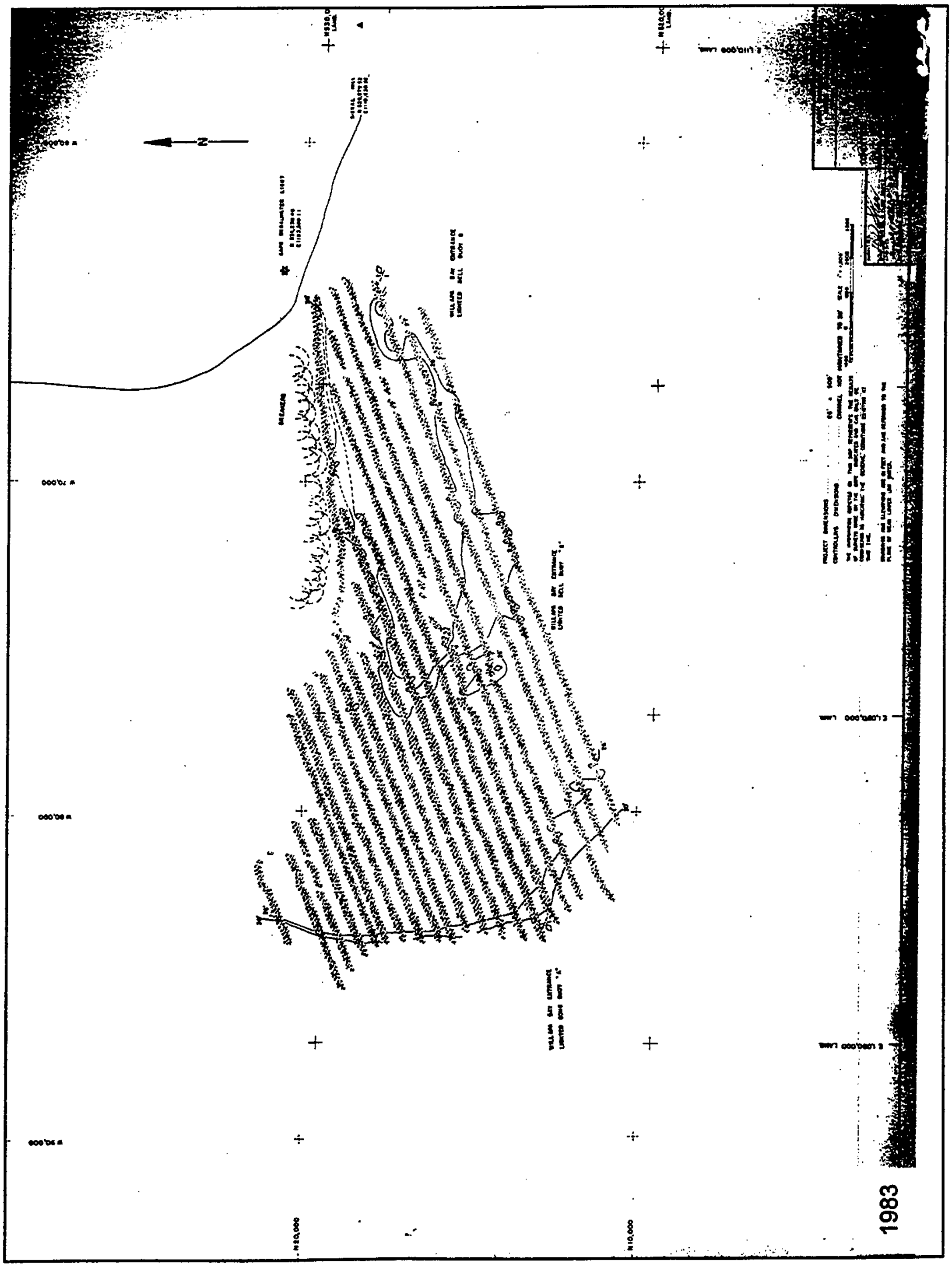

U.S. ARMY
CORPS OF ENGINEERS
WILLAPA HARBOR
WASHINGTON
BAR AND ENTRANCE
LEGEND
PACIFIC
OCEAN
1977

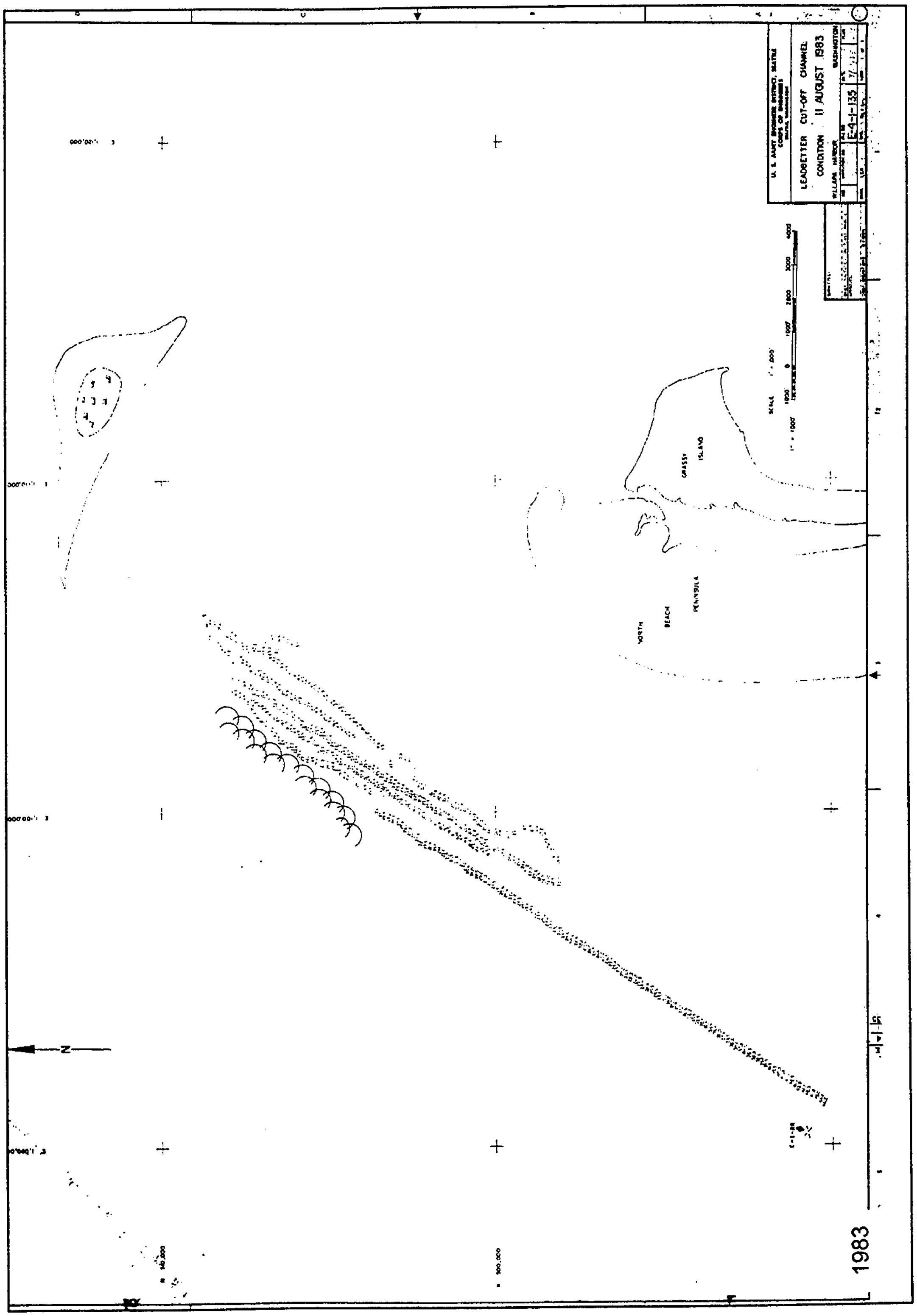

WILLAPA HARBOR
WASHINGTON
BAR AND ENTRANCE
CONDITION JULY 8 AUGUST 1978
Seattle District, Seattle, Wash. 8 January 1979
LEGEND
CAPE SHOALWATER
BREAKERS
PACIFIC
OCEAN
1978


WILLAPA RIVER AND HARBOR
WASHINGTON
BAR CHANNEL
CONDITION
16 APRIL 1981
18 MAY 1981
1981

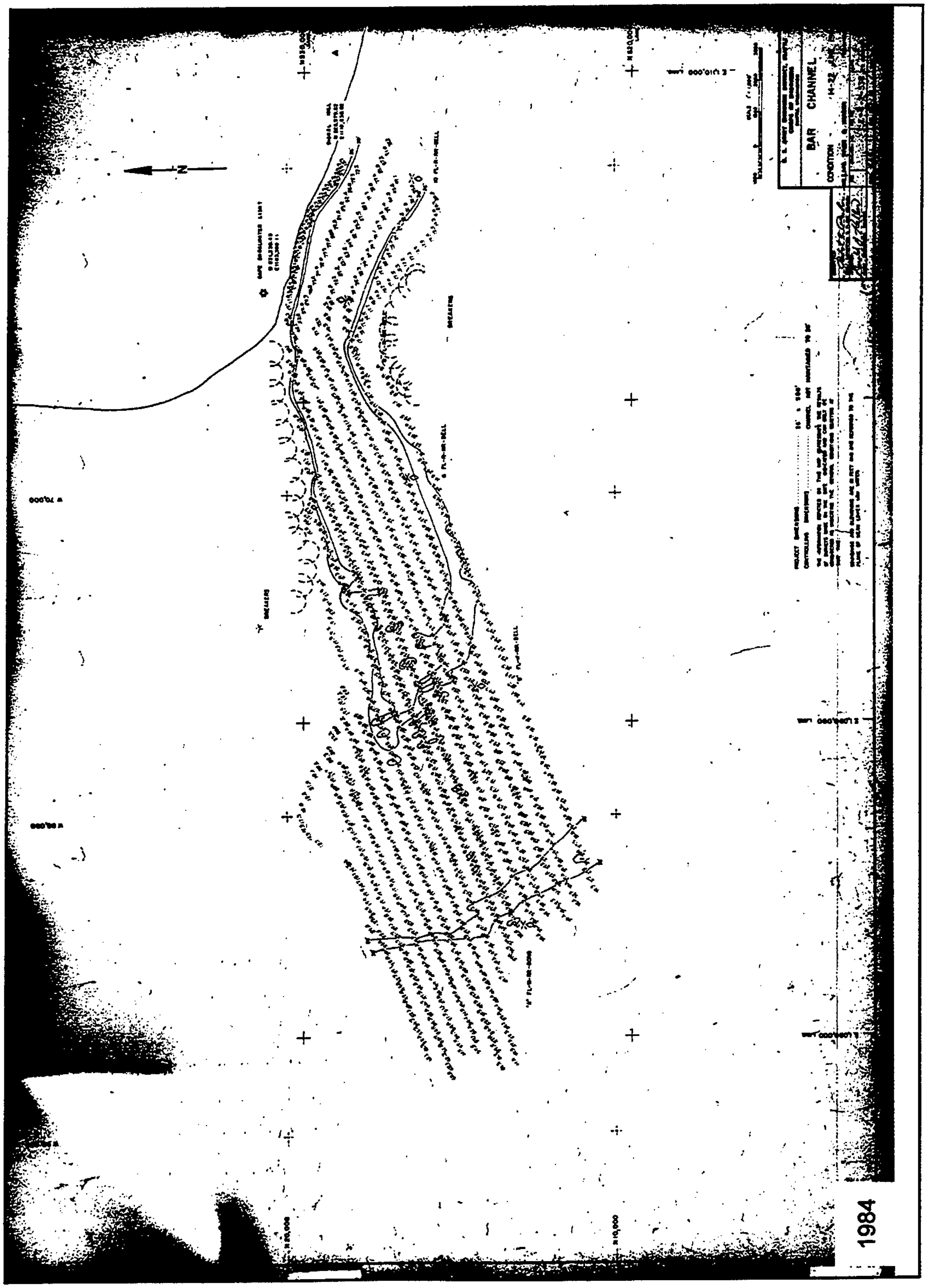

LEADBETTER CUT-OFF CHANNEL
CONDITION MAY 1982
WILLAPA HARBOR
GRASSY ISLAND
NORTH BEACH PENINSULA
1982

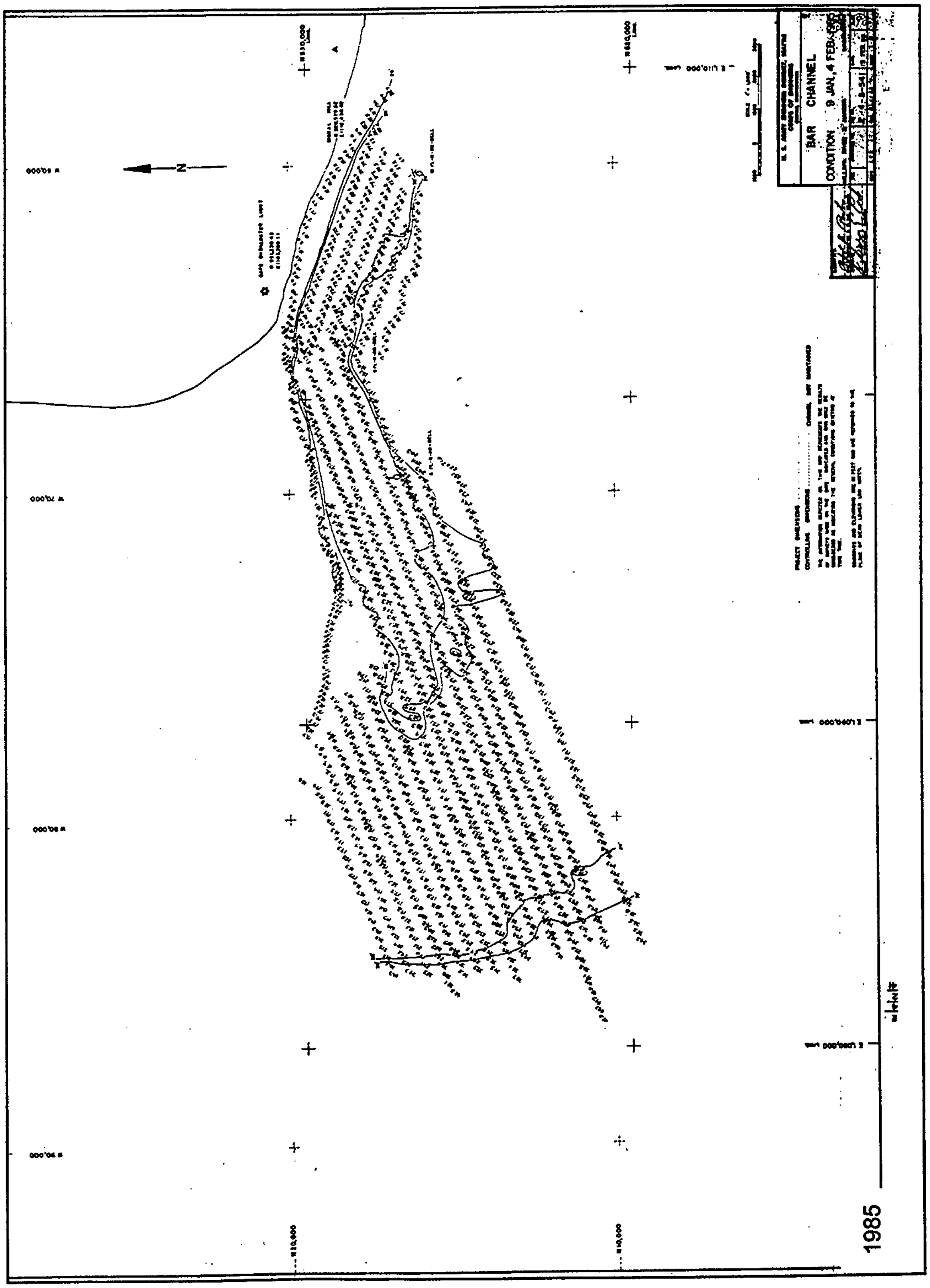

BAR CHANNEL
1982

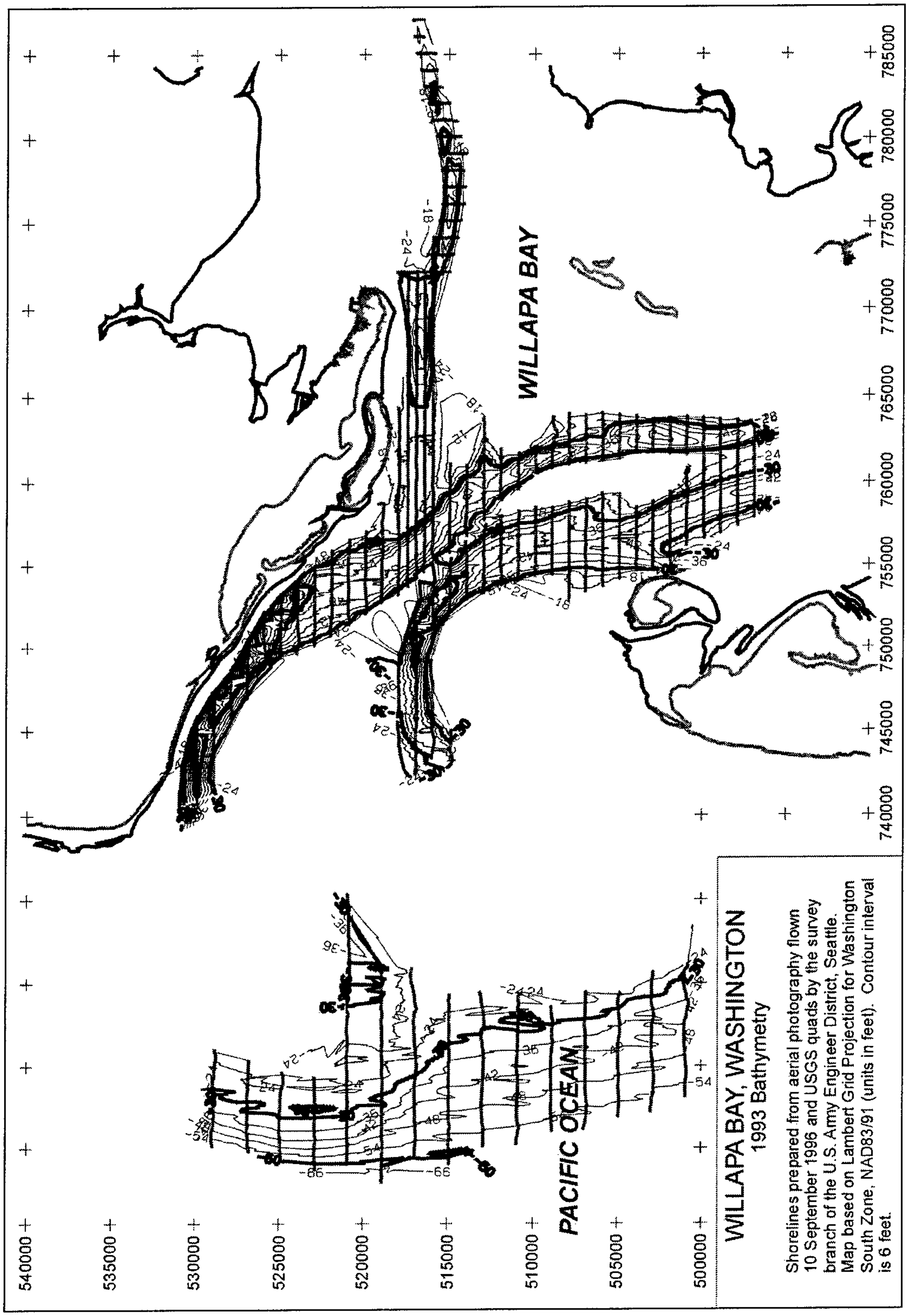

1983



LEADBETTER CUT-OFF CHANNEL
CONDITION 11 AUGUST 1983
WILLAPA HARBOR
WASHINGTON
GRASSY ISLAND
NORTH BEACH PENINSULA
1983

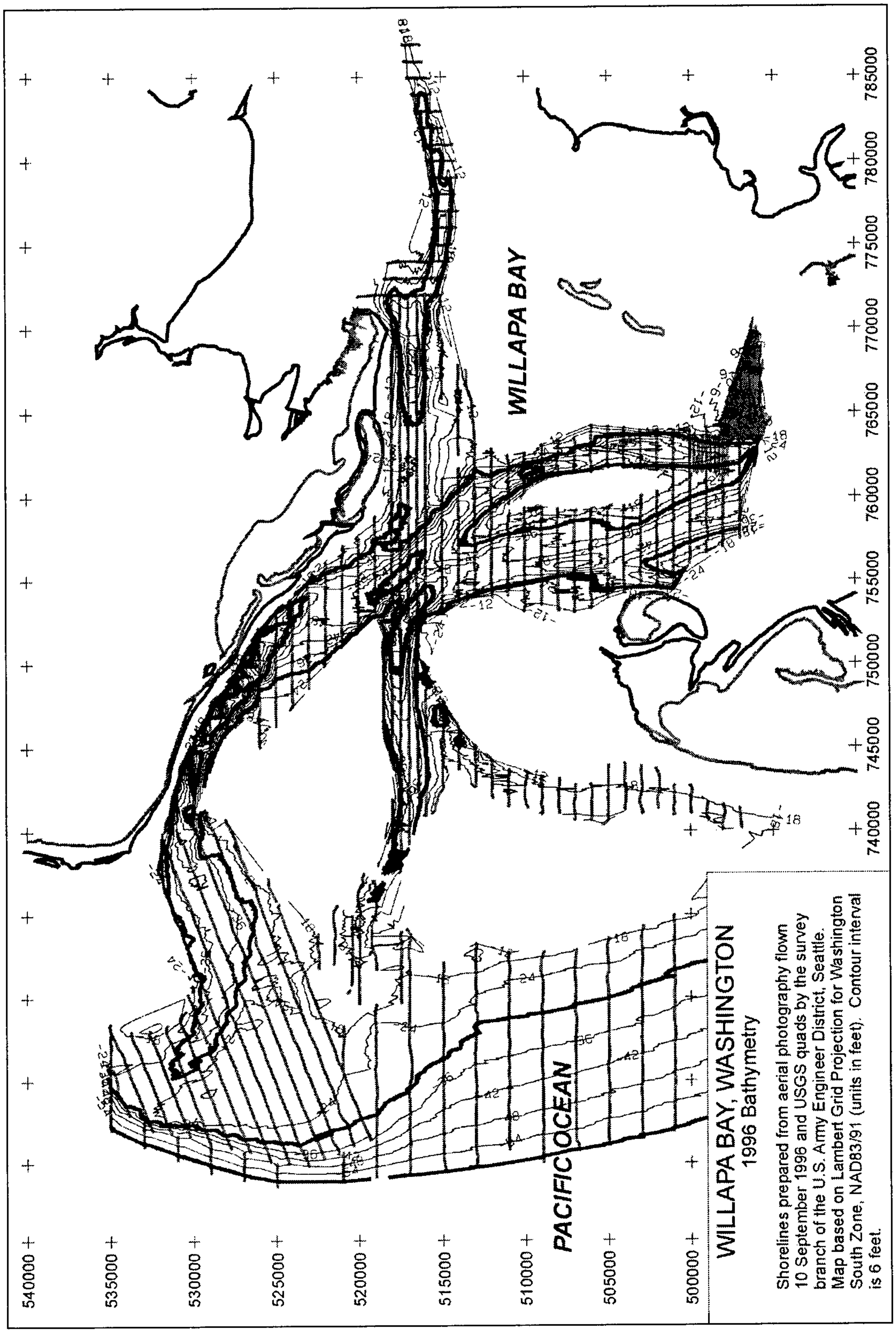

BAR CHANNEL
1984

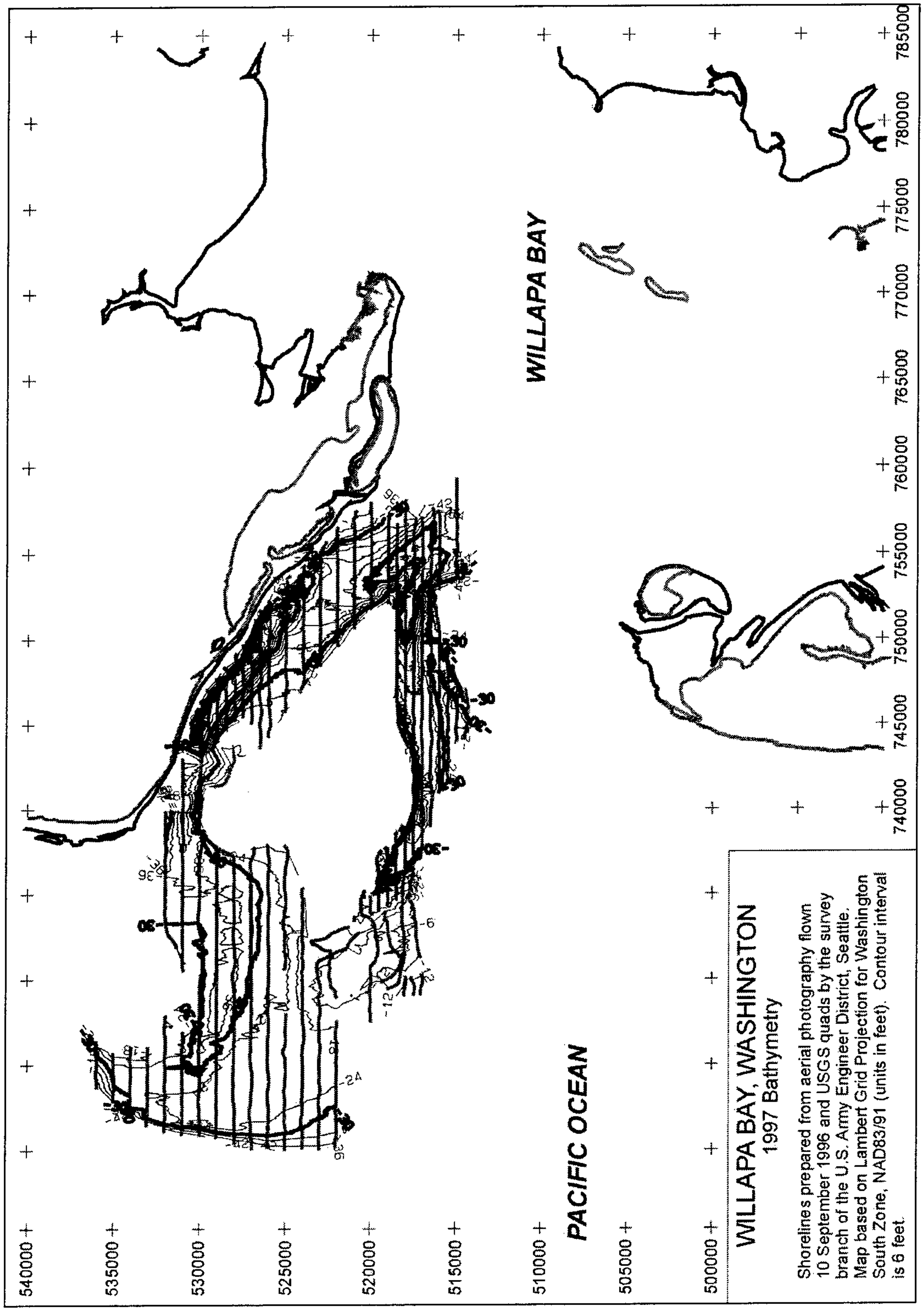

BAR CHANNEL
1985

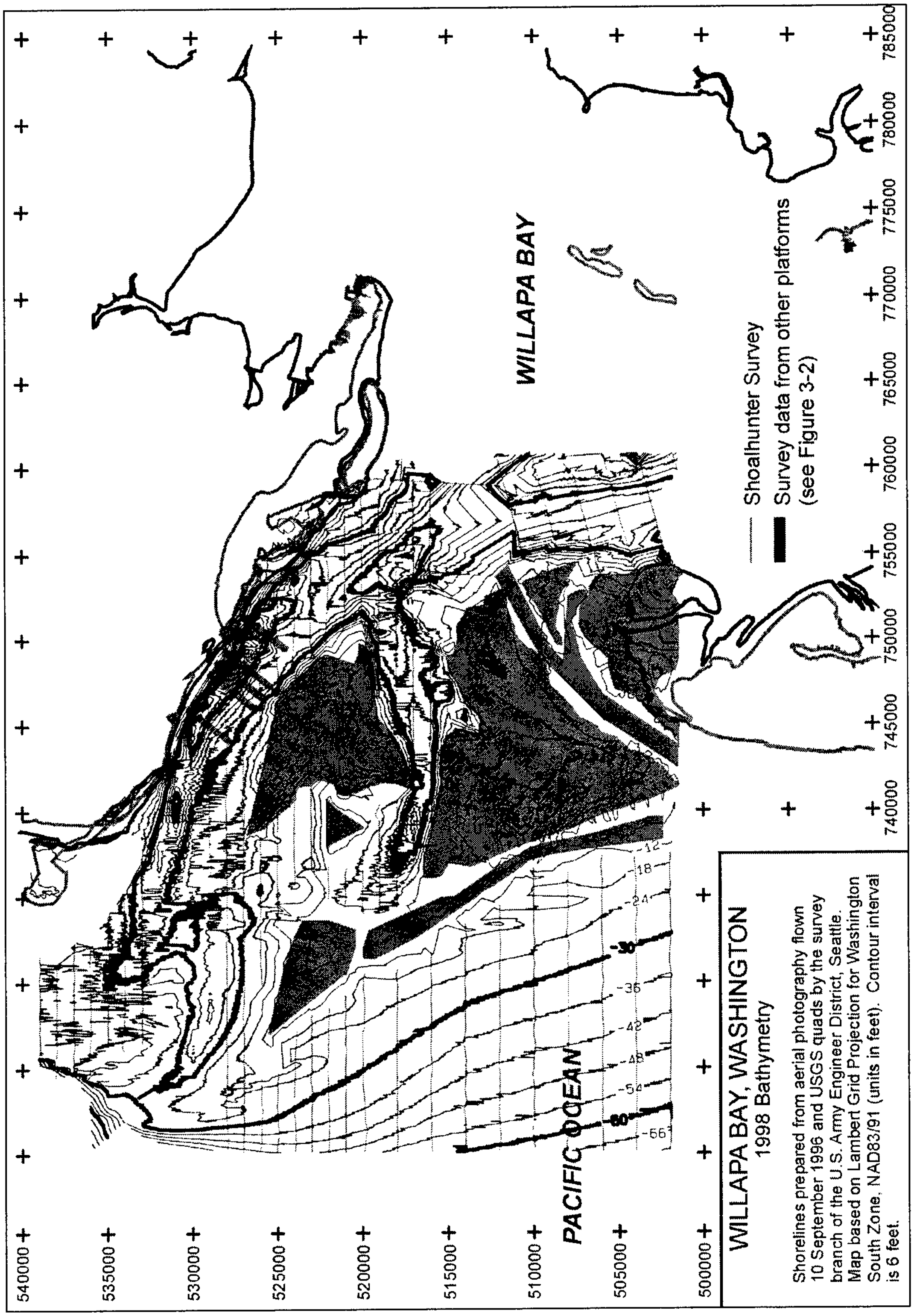


WILLAPA BAY, WASHINGTON
1993 Bathymetry

Shorelines prepared from aerial photography flown 10 September 1996 and USGS quads by the survey branch of the U.S. Army Engineer District, Seattle. Map based on Lambert Grid Projection for Washington South Zone, NAD83/91 (units in feet). Contour interval is 6 feet.


WILLAPA BAY
PACIFIC OCEAN
WILLAPA BAY, WASHINGTON
1996 Bathymetry
Shorelines prepared from aerial photography flown 10 September 1996 and USGS quads by the survey branch of the U.S. Army Engineer District, Seattle. Map based on Lambert Grid Projection for Washington South Zone, NAD83/91 (units in feet). Contour interval is 6 feet.
540000
535000
530000
525000
520000
515000
510000
505000
500000
740000
745000
750000
755000
760000
765000
770000
775000
780000
785000


WILLAPA BAY, WASHINGTON
1997 Bathymetry
Shorelines prepared from aerial photography flown 10 September 1996 and USGS quads by the survey branch of the U.S. Army Engineer District, Seattle. Map based on Lambert Grid Projection for Washington South Zone, NAD83/91 (units in feet). Contour interval is 6 feet.
WILLAPA BAY
PACIFIC OCEAN


WILLAPA BAY, WASHINGTON
1998 Bathymetry
Shorelines prepared from aerial photography flown 10 September 1996 and USGS quads by the survey branch of the U.S. Army Engineer District, Seattle. Map based on Lambert Grid Projection for Washington South Zone, NAD83/91 (units in feet). Contour interval is 6 feet.
Shoalhunter Survey
Survey data from other platforms (see Figure 3-2)
WILLAPA BAY
PACIFIC OCEAN
540000
535000
530000
525000
520000
515000
510000
505000
500000
740000
745000
750000
755000
760000
765000
770000
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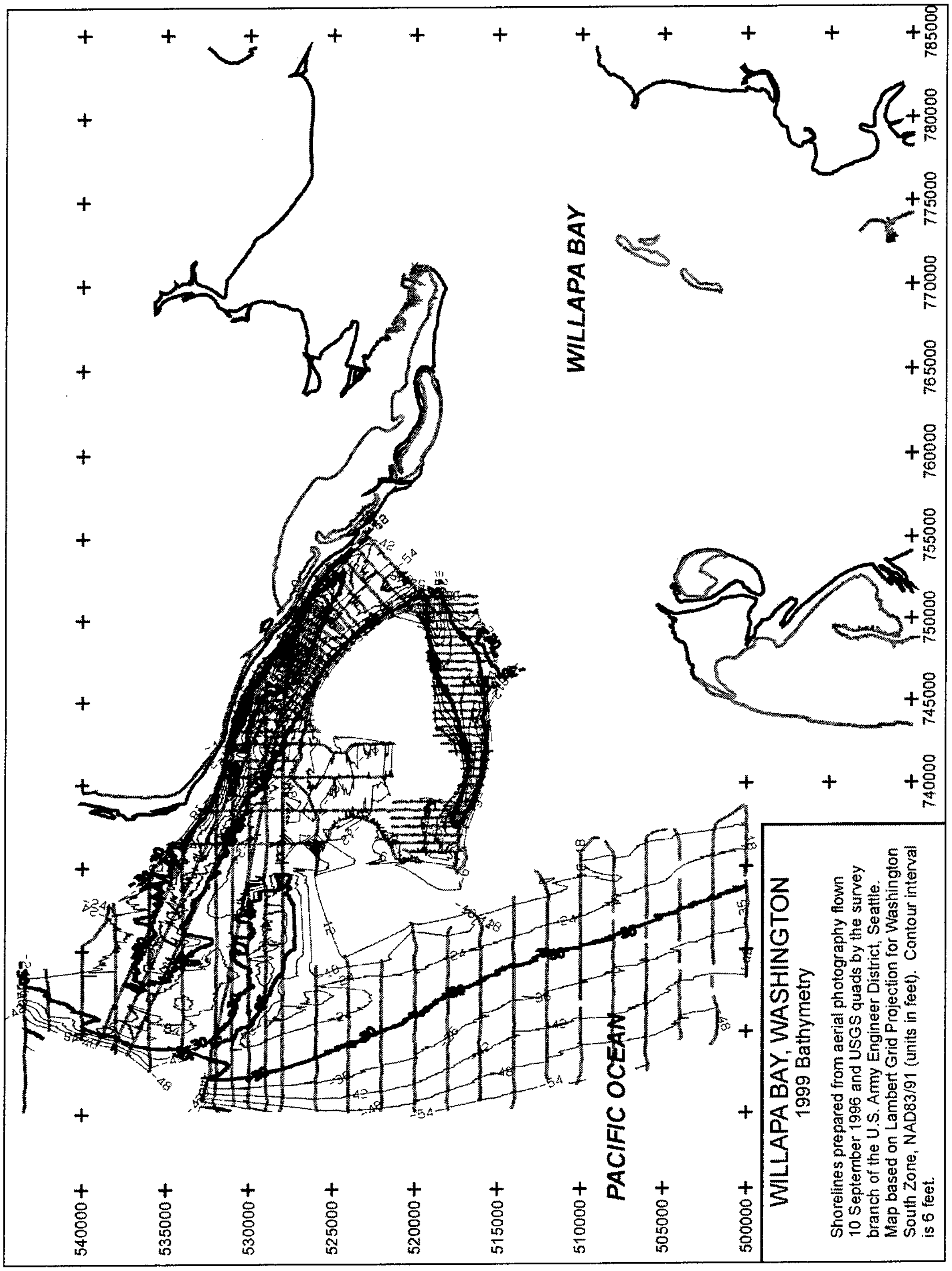
-48
-54
-60
-66



WILLAPA BAY, WASHINGTON
1999 Bathymetry

Shorelines prepared from aerial photography flown 10 September 1996 and USGS quads by the survey branch of the U.S. Army Engineer District, Seattle. Map based on Lambert Grid Projection for Washington South Zone, NAD83/91 (units in feet). Contour interval is 6 feet.

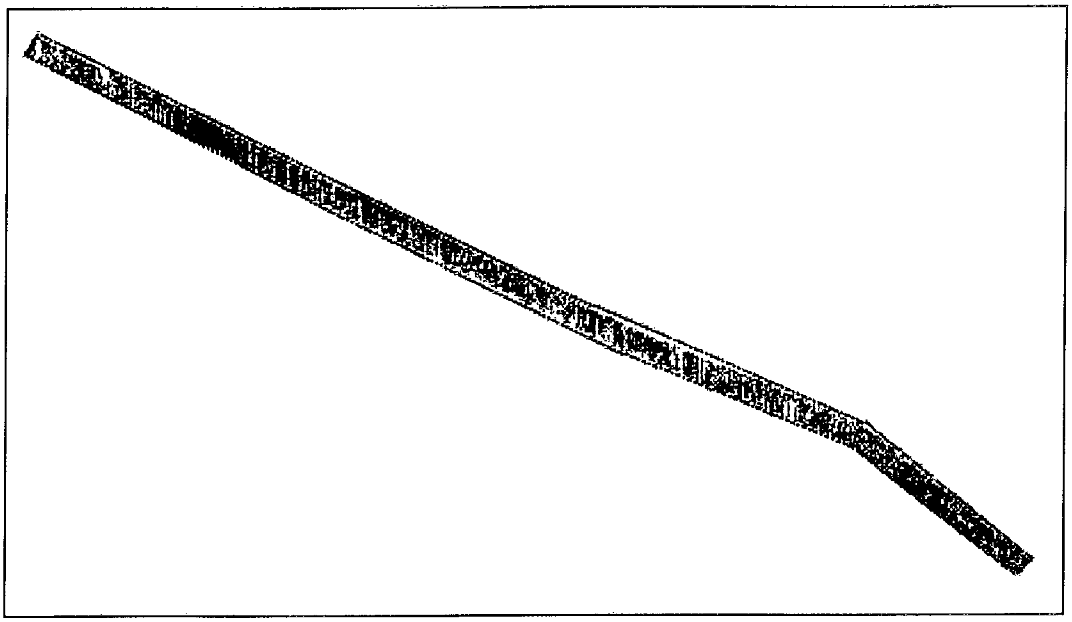
Appendix B[1]
Template Layouts and Recent Volume Changes

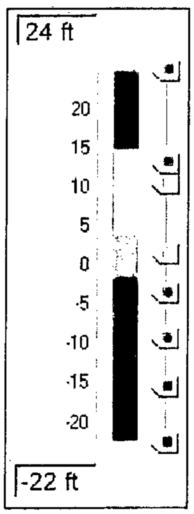
Figures in this appendix show positions within the bay entrance of seven templates corresponding to the three categories of channel alternatives that passed screening as described in Chapters 2 and 7 of the main text. A template is a geometric form that matches the channel design cross section (width, depth, and side slopes). The volumes of erosion (cut) and deposition (fill) within the templates between the 1998 and 1999 surveys are tabulated. Average, maximum, and minimum thicknesses and the spatial distribution of changes are shown in tables and graphs.

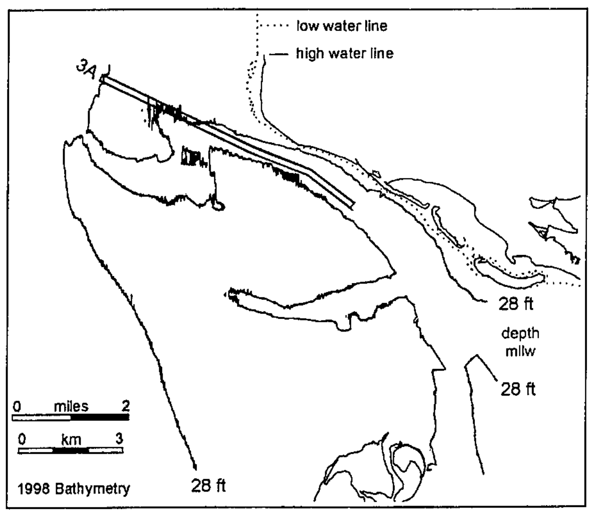
This appendix summarizes 1998-99 channel changes within templates covered by dense soundings that were collected concurrently with extensive process measurements. The numerical simulations presented in Chapter 6 are also for this year. Because changes from one year to the next can vary widely, Chapter 3 combined 1998-to-1999 changes with statistics from other years (Table 3-4 and 3-5) and other templates (Figure 3-22) to derive the likely long-term cumulative dredging requirement for each channel alternative (Table 3-7). Depths are referenced to mean lower low water (mllw).

[1] Written by Mr. Edward B. Hands, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory (CHL), Vicksburg, MS. Ms. Mary C. Allison (CHL) prepared the digital images.

North Channel Option
Template 3A

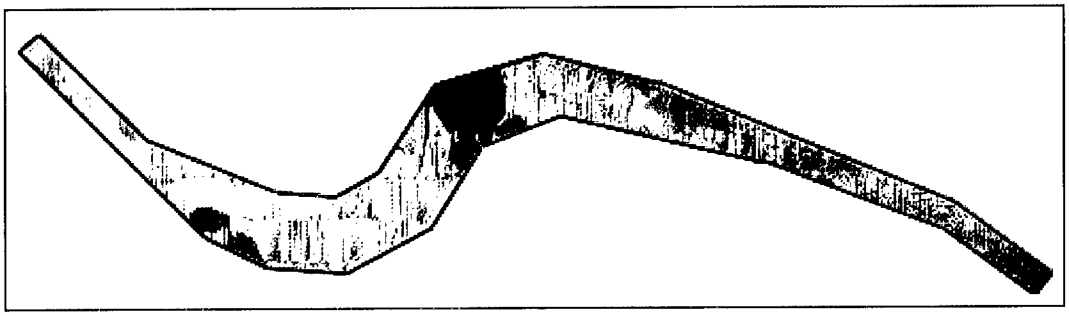


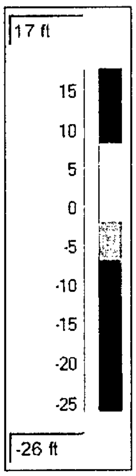


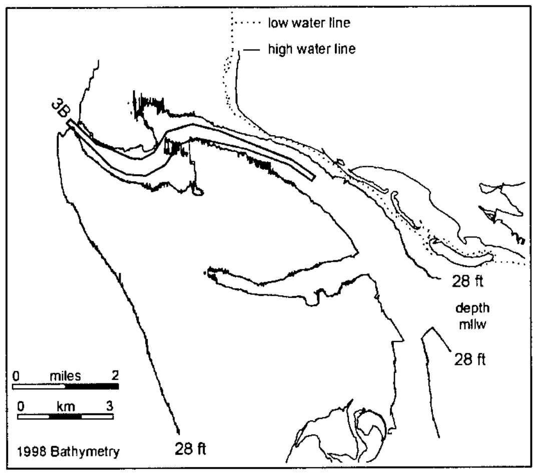


Above or Below Design Depth (28 ft mllw)	8-Month Change 8/98 to 4/99			Annualized Change Rate			
				Fill		Net Change	
	Fill Cu yd	Cut cu yd	Net Change cu yd	cu yd	cu ft/sq ft	cu yd	cu ft/sq ft
Above Design Depth	3.7E+04	-8.0E+05	-7.6E+05	5.2E+04	0.1	-1.1E+06	-1.7
Below Design Depth	1.4E+05	-3.8E+06	-3.7E+06				
Total	1.7E+05	-4.6E+06	-4.5E+06	2.5E+05	0.4	-6.3E+06	-9.9
NOTE: Template area = 17,168,038 sq ft.							

North Channel Option
Template 3B



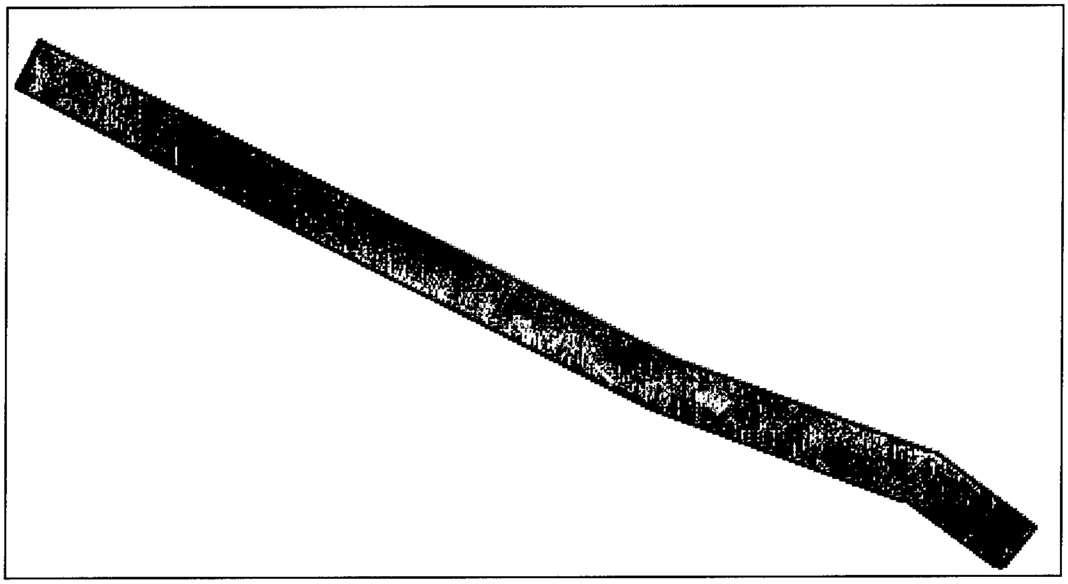


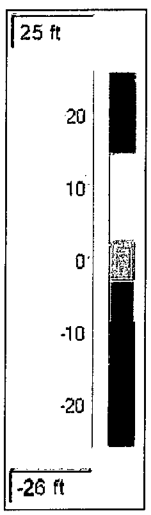


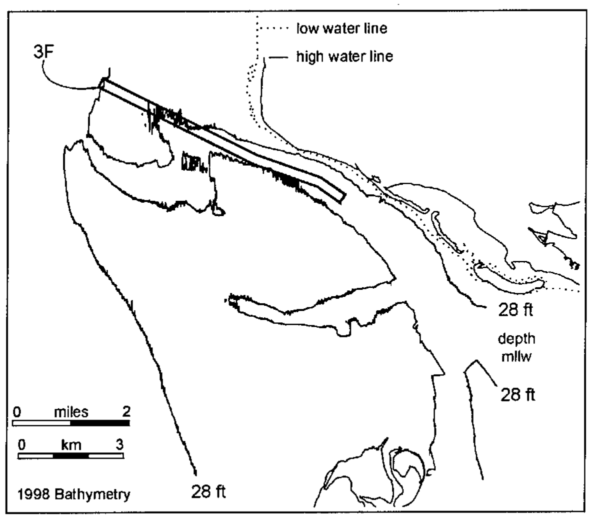
Though natural channels were smaller than design alternatives, the calculated changes shown for the other pages in this appendix are indicative of the shoaling potential during 1998-1999. For the migrating Alternative 3B, however, the changes within Template 3B do not represent either long-term changes or more importantly any specific dredging requirement because, if Alternative 3B had been present, unacceptably large-volume dredging would probably have been scheduled in 1998 to relocate the authorized channel to a more northerly position close to Alternative 3A. The changes shown here do represent what would have occurred during 1998-1999 within Template 3B. These measured changes compare well with modeled current and deposition patterns for the same area based on simulated conditions for shorter periods during this same year (Chapter 6 and Appendix G).

Above or Below Design Depth (28 ft mllw)	8-Month Change 8/98 to 4/99			Annualized Change Rate			
				Fill		Net Change	
	Fill cu yd	Cut cu yd	Net Change cu yd	cu yd	cu ft/sq ft	cu yd	cu ft/sq ft
Above Design Depth	1.0E+06	-3.4E+03	1.0E+06	1.4E+06	1.2	1.4E+06	1.2
Below Design Depth	2.4E+06	-2.0E+06	4.3E+05				
Total	3.4E+06	-2.0E+06	1.4E+06	4.9E+06	1.1	2.0E+06	1.6
NOTE: Template area = 33,155,116 sq ft.							

North Channel Option
Template 3F

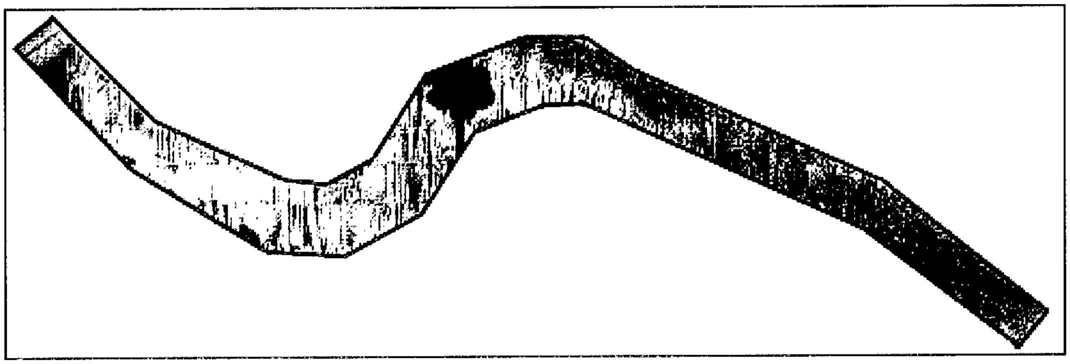


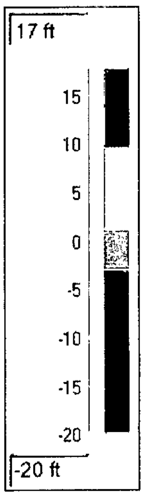


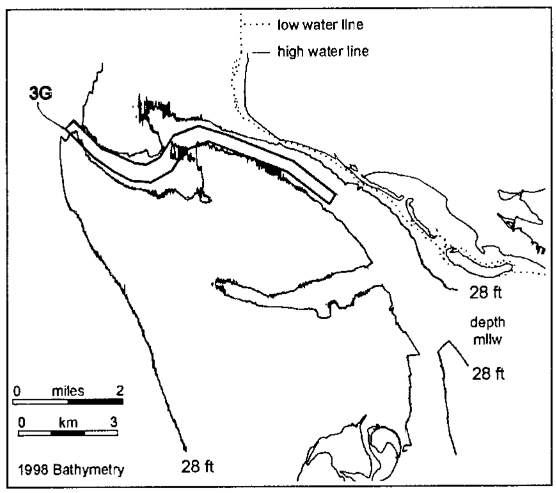


Above or Below Design Depth (38 ft mllw)	**8-Month Change 8/98 to 4/99**			**Annualized Change Rate**			
				Fill		**Net Change**	
	Fill cu yd	**Cut cu yd**	**Net Change cu yd**	**cu yd**	**cu ft/sq ft**	**cu yd**	**cu ft/sq ft**
Above Design Depth	3.1E+05	-3.2E+06	-2.9E+06	4.3E+05	12.7	-4.1E+06	-121.3
Below Design Depth	1.3E+05	-2.3E+06	-2.2E+06				
Total	4.3E+05	-5.5E+06	-5.1E+06	6.1E+05	17.9	-7.2E+06	-212.0
NOTE: Template area = 24,499,897 sq ft.							

North Channel Option
Template 3G



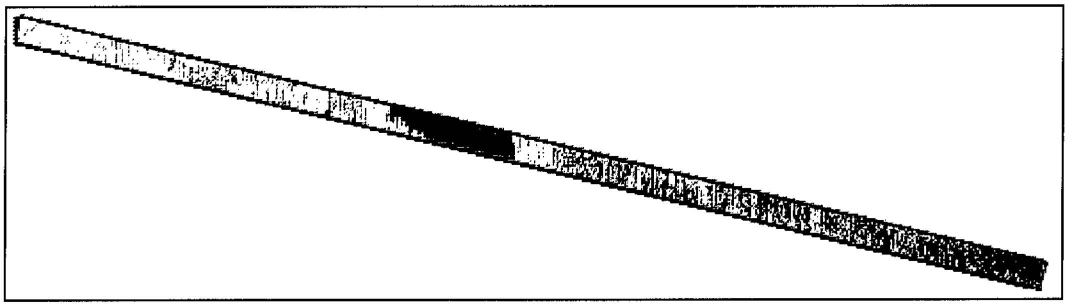


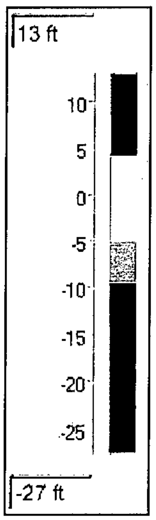


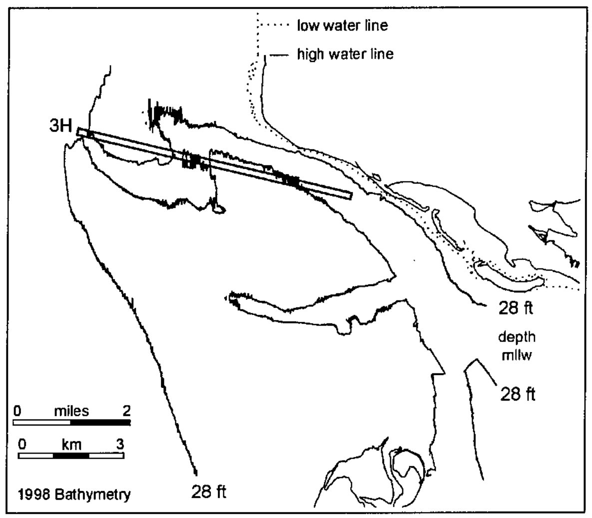
Though natural channels were smaller than design alternatives, the calculated changes shown for the other pages in this appendix are indicative of the shoaling potential for those Alternatives during 1998-1999. For the migrating Alternative 3G, the changes within Template 3G do not represent either long-term changes or more importantly any specific dredging requirement because, if Alternative 3G had been in effect, relocation dredging would probably have been scheduled in 1998 to move the authorized channel to a more northerly position close to Template 3F. The changes shown here do represent what would have occurred during 1998-1999 within Template 3G. These changes compare well with modeled current and deposition patterns for the same area, based on simulated conditions for shorter periods during the same year (Appendix 3G).

Above or Below Design Depth (38 ft mllw)	8-Month Change 8/98 to 4/99			Annualized Change Rate			
	Fill	Cut	Net Change	Fill		Net Change	
	cu yd	cu yd	cu yd	cu yd	cu ft/sq ft	cu yd	cu ft/sq ft
Above Design Depth	3.5E+06	-7.5E+05	2.7E+06	4.9E+06	3.0	3.9E+06	2.4
Below Design Depth	3.7E+05	-3.3E+06	-2.9E+06				
Total	3.8E+06	-4.0E+06	-1.6E+05	5.4E+06	3.4	-2.3E+05	-0.1
NOTE: Template area = 43,596,918 sq ft.							

SR-105 Channel Options 3Ha and 3Hb
Template 3H

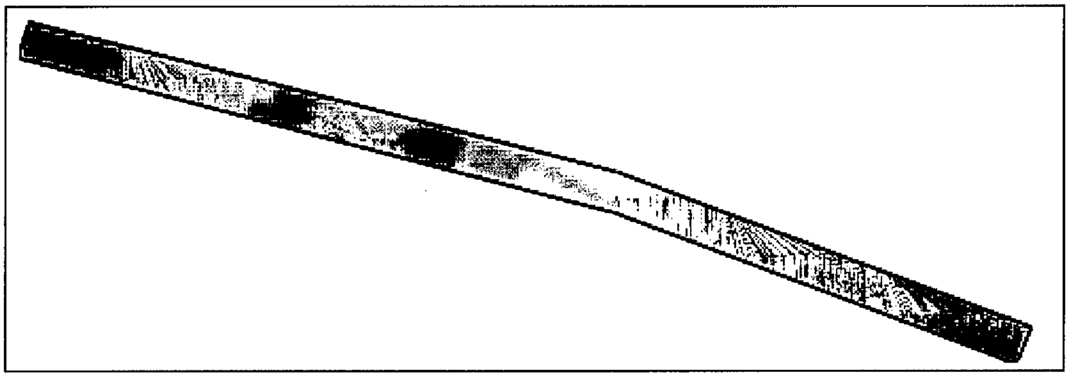


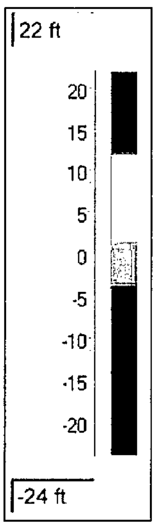


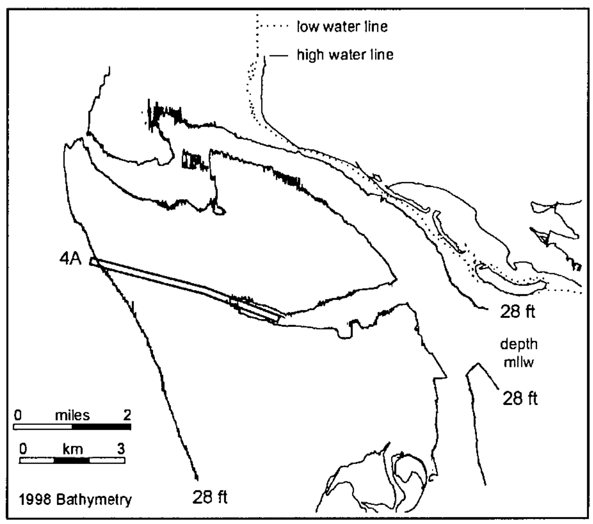


Above or Below Design Depth (28 ft mllw)	8-Month Change 8/98 to 4/99			Annualized Change Rate			
				Fill		Net Change	
	Fill cu yd	Cut cu yd	Net Change cu yd	cu yd	cu ft/sq ft	cu yd	cu ft/sq ft
Above Design Depth	5.7E+05	-1.0E+06	-4.5E+05	8.0E+05	1.3	-6.3E+05	-1.0
Below Design Depth	1.1E+05	-1.7E+06	-1.5E+06				
Total	6.8E+05	-2.7E+06	-2.0E+06	9.6E+05	1.5	-2.8E+06	-4.4
NOTE: Template area = 17,061,317 sq ft.							

Middle Channel Option
Template 4A

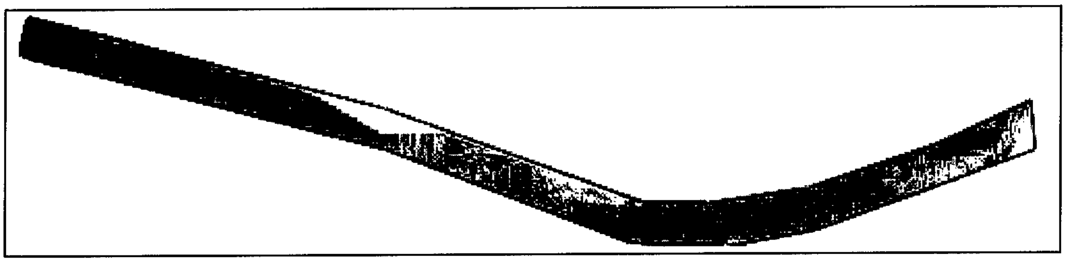


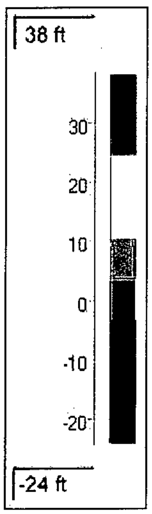


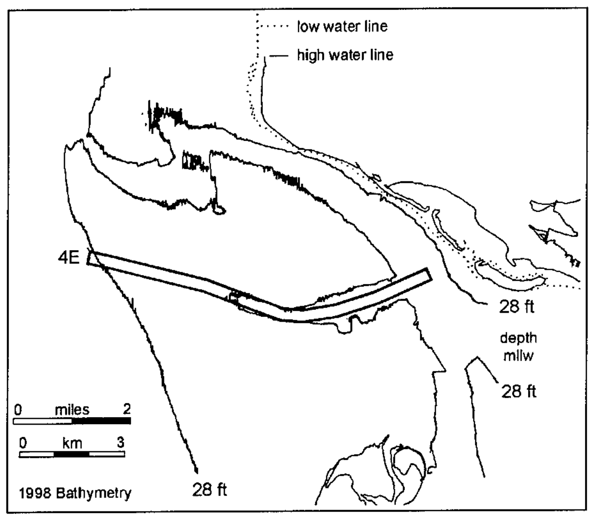


Above or Below Design Depth (28 ft mllw)	8-Month Change 8/98 to 4/99			Annualized Change Rate			
				Fill		Net Change	
	Fill cu yd	Cut cu yd	Net Change cu yd	cu yd	cu ft/sq ft	cu yd	cu ft/sq ft
Above Design Depth	2.9E+05	-2.4E+05	5.1E+04	4.1E+05	0.9	7.2E+04	0.2
Below Design Depth	4.0E+05	-4.5E+05	-5.3E+04				
Total	6.9E+05	-6.9E+05	-1.7E+03	9.7E+05	2.2	-2.4E+03	0.0
NOTE: Template area = 11,923,267 sq ft.							

Middle Channel Option
Template 4E



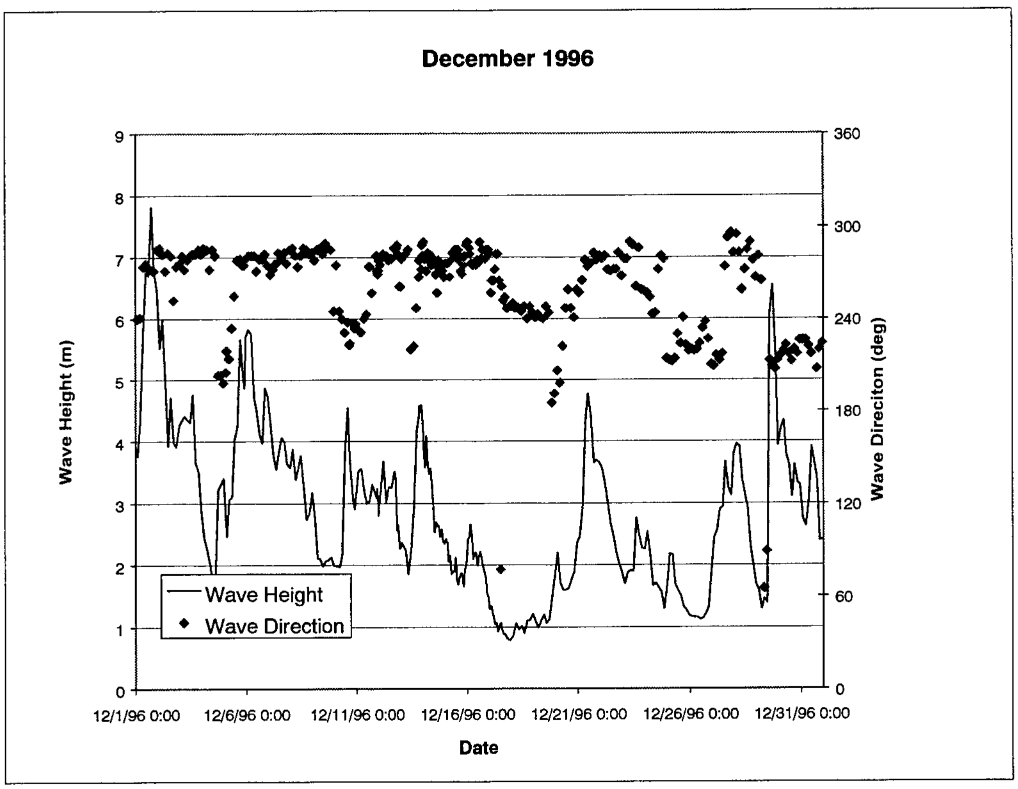




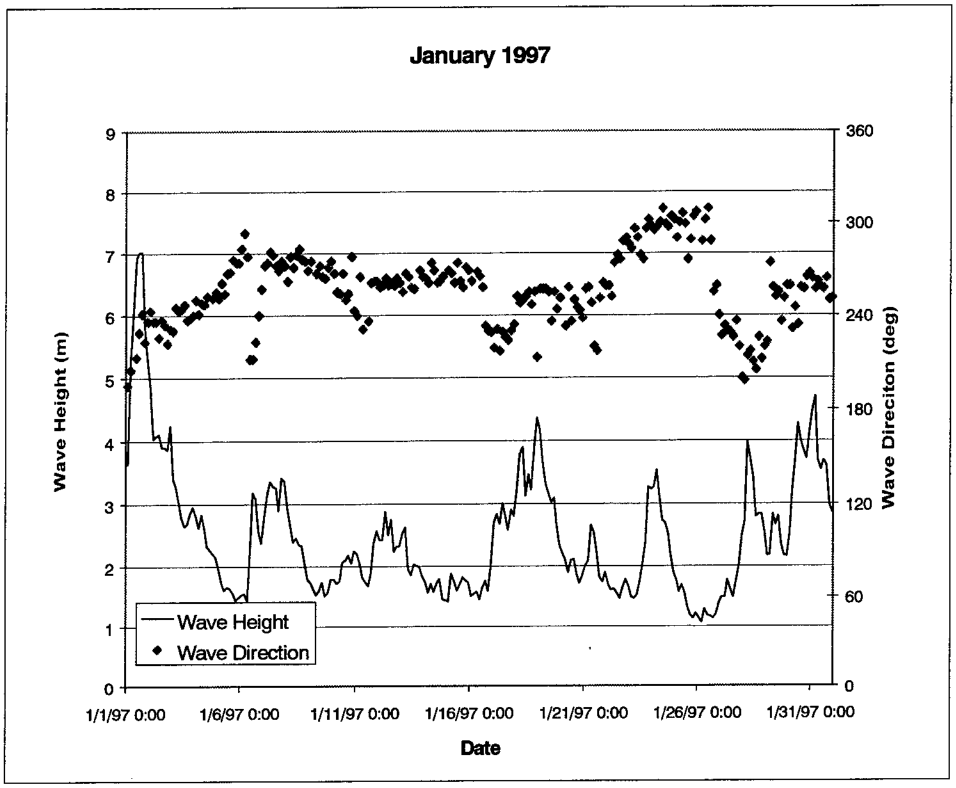
Above or Below Design Depth (38 ft mllw)	8-Month Change 8/98 to 4/99			Annualized Change Rate			
				Fill		Net Change	
	Fill cu yd	Cut cu yd	Net Change cu yd	cu yd	cu ft/sq ft	cu yd	cu ft/sq ft
Above Design Depth	1.6E+06	-1.7E+06	-9.0E+04	2.2E+06	1.5	-1.3E+05	-0.1
Below Design Depth	4.1E+05	-1.1E+06	-6.8E+05				
Total	2.0E+06	-2.8E+06	-7.7E+05	2.8E+06	1.9	-1.1E+06	-0.7
NOTE: Template area = 39,797,130 sq ft.							

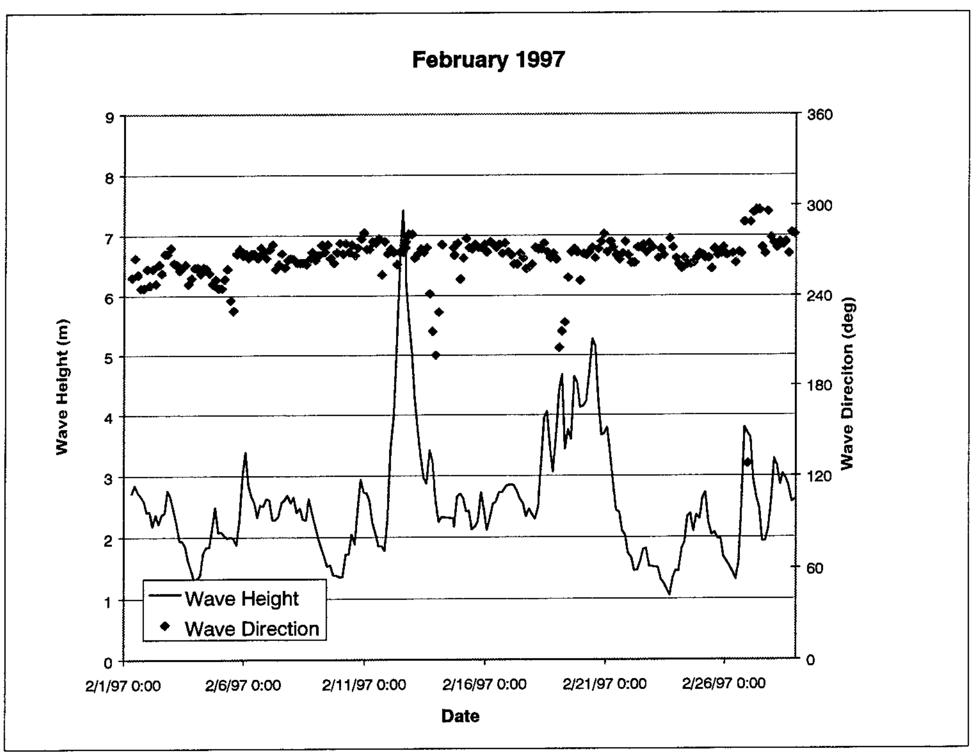
Appendix C
Winter-Wave Time Histories[1]

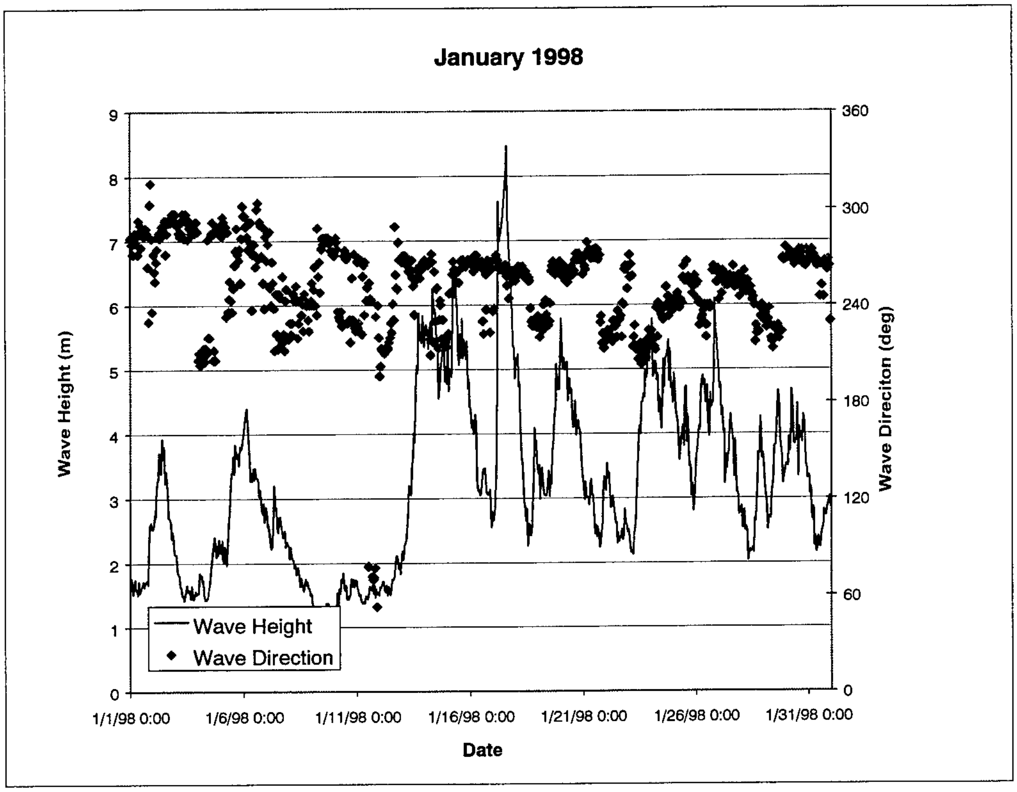
This appendix includes plots of 5 months of winter waves measured at the Grays Harbor, WA, buoy (December 1996 - February 1998). The average winter wave height is 3 m, and the wave height exceeds 3 m for 42 percent of the winter. A wave height of 3 m is approximately the upper limit for navigating the Willapa Bay entrance.

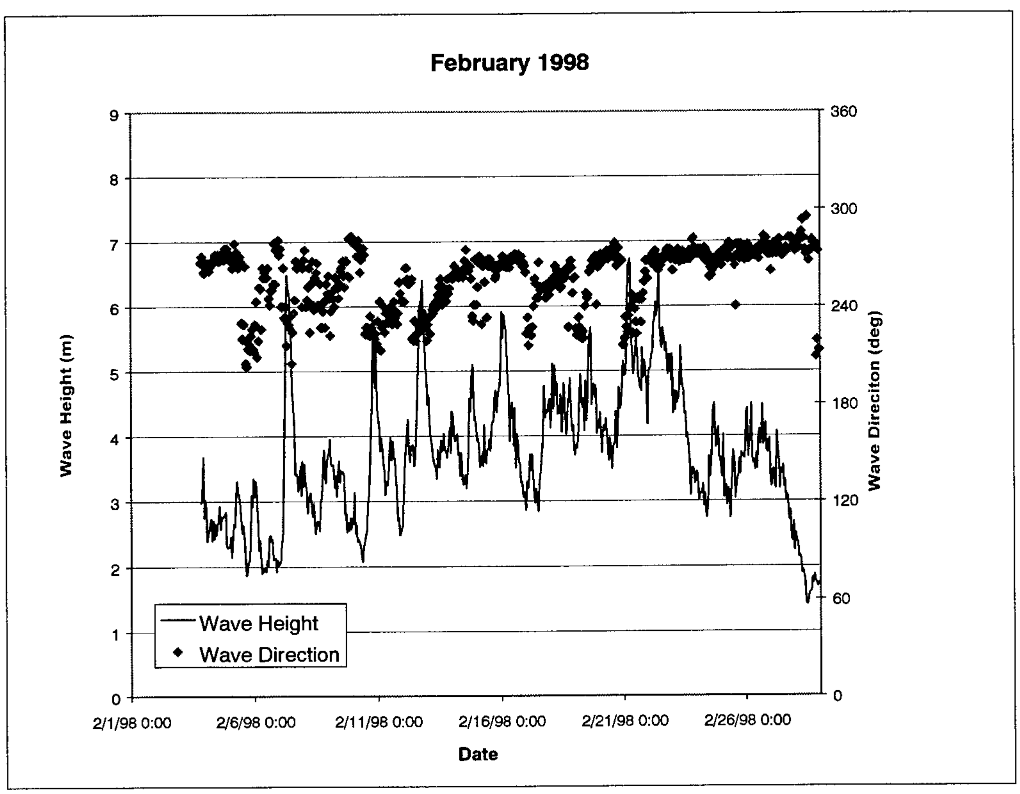


[1] Written by Dr. Jane McKee Smith, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS.


January 1997
Wave Height (m)
Wave Direciton (deg)
Date
1/1/97 0:00
1/6/97 0:00
1/11/97 0:00
1/16/97 0:00
1/21/97 0:00
1/26/97 0:00
1/31/97 0:00
Wave Height
Wave Direction


February 1997
Wave Height (m)
Wave Direciton (deg)
Date
2/1/97 0:00
2/6/97 0:00
2/11/97 0:00
2/16/97 0:00
2/21/97 0:00
2/26/97 0:00
Wave Height
Wave Direction


January 1998
Wave Height (m)
Wave Direciton (deg)
Date
0
1
2
3
4
5
6
7
8
9
0
60
120
180
240
300
360
1/1/98 0:00
1/6/98 0:00
1/11/98 0:00
1/16/98 0:00
1/21/98 0:00
1/26/98 0:00
1/31/98 0:00
Wave Height
Wave Direction

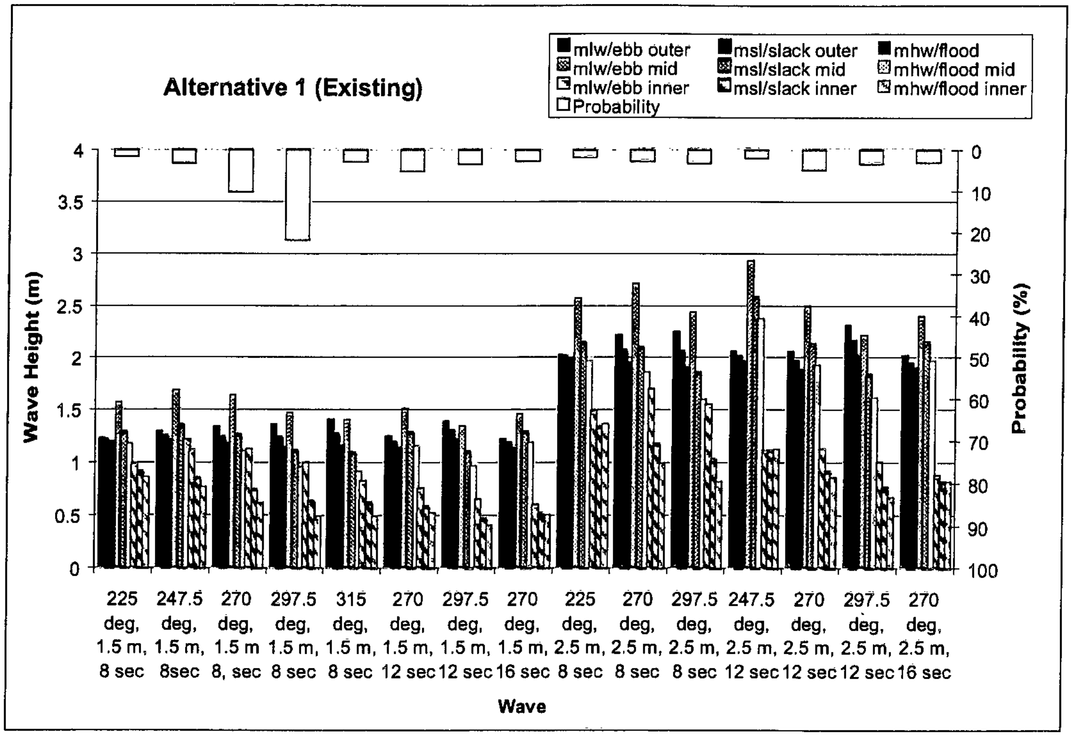

February 1998
Wave Height (m)
Wave Direciton (deg)
Date
0
1
2
3
4
5
6
7
8
9
0
60
120
180
240
300
360
2/1/98 0:00
2/6/98 0:00
2/11/98 0:00
2/16/98 0:00
2/21/98 0:00
2/26/98 0:00
Wave Height
Wave Direction

Appendix D
Representative Wave Plots[1]

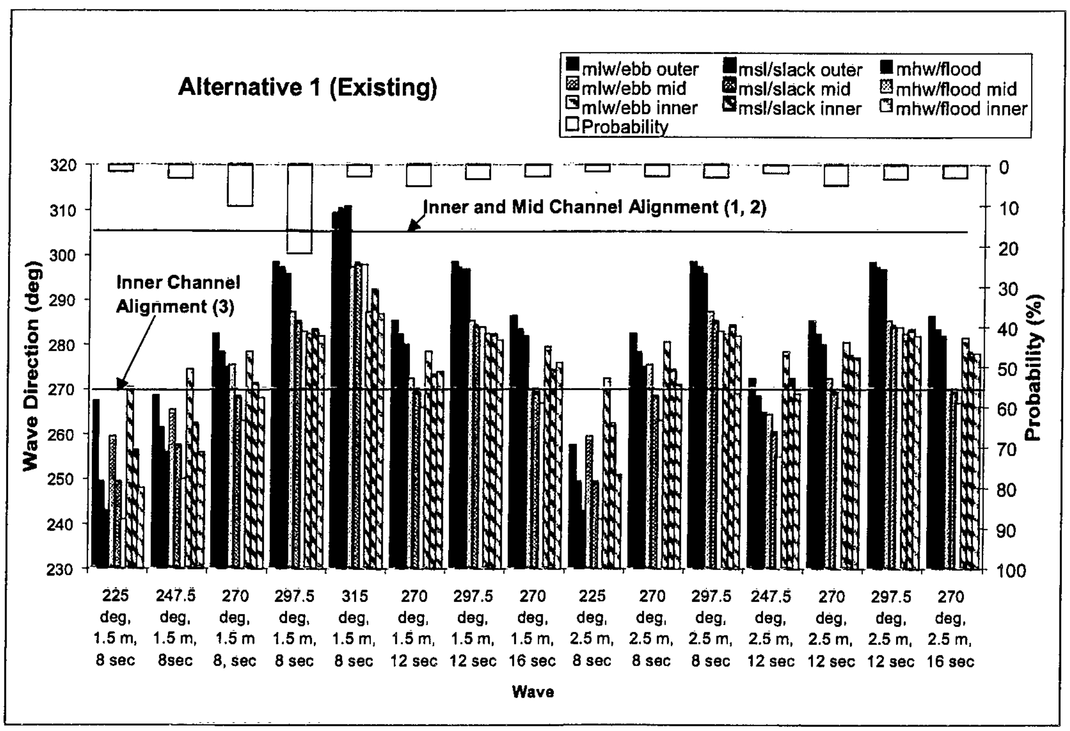
This appendix includes plots of the transformation of representative waves calculated by the STWAVE model as described in Chapter 5. Fifteen waves were transformed for channel Alternatives 1 (existing condition), 3A, 3B, 4A, and 3H. The plots for each alternative appear on a single page with wave heights in the top plot and wave directions in the bottom plot. For each plot, the 15 incident wave conditions are listed across the bottom. Across the top of the plot are bars giving the probability of occurrence (reference to the right-hand scale). For each wave condition, nine bars represent three locations in the channel (outer, middle, and inner) and three typical tides/currents (mean low water (mlw)/ebb, mean sea level (msl)/slack, and mean high water (mhw)/flood). The channel alignments are plotted as horizontal lines for reference.

For Alternatives 1 and 3A, the outer channel and midchannel alignment is approximately 305 deg and the inner channel alignment is 270 deg. For Alternative 4A, the channel is straight with an alignment of 270 deg throughout the channel. The channel alignment for Alternative 3B is sinuous. The alignment is approximately 300 deg in the outer channel, 270 deg in the midchannel, and 230 deg in the inner channel. The channel alignment for Alternative 3H is 281 deg.

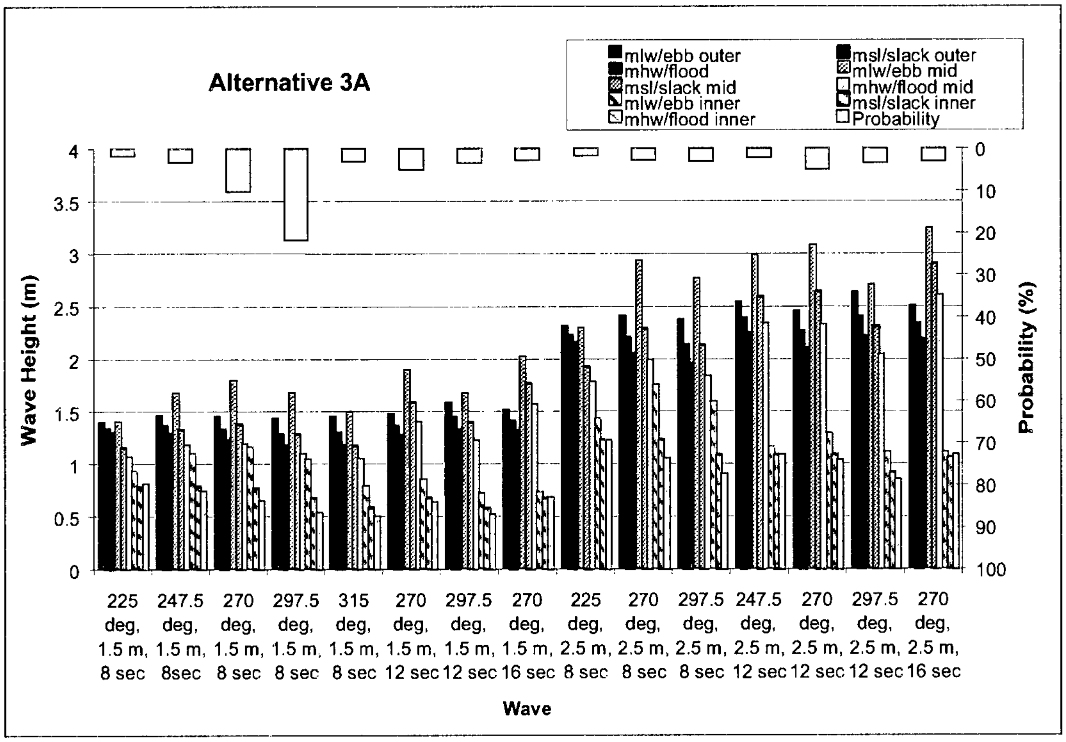
[1] Written by Dr. Jane McKee Smith, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS.

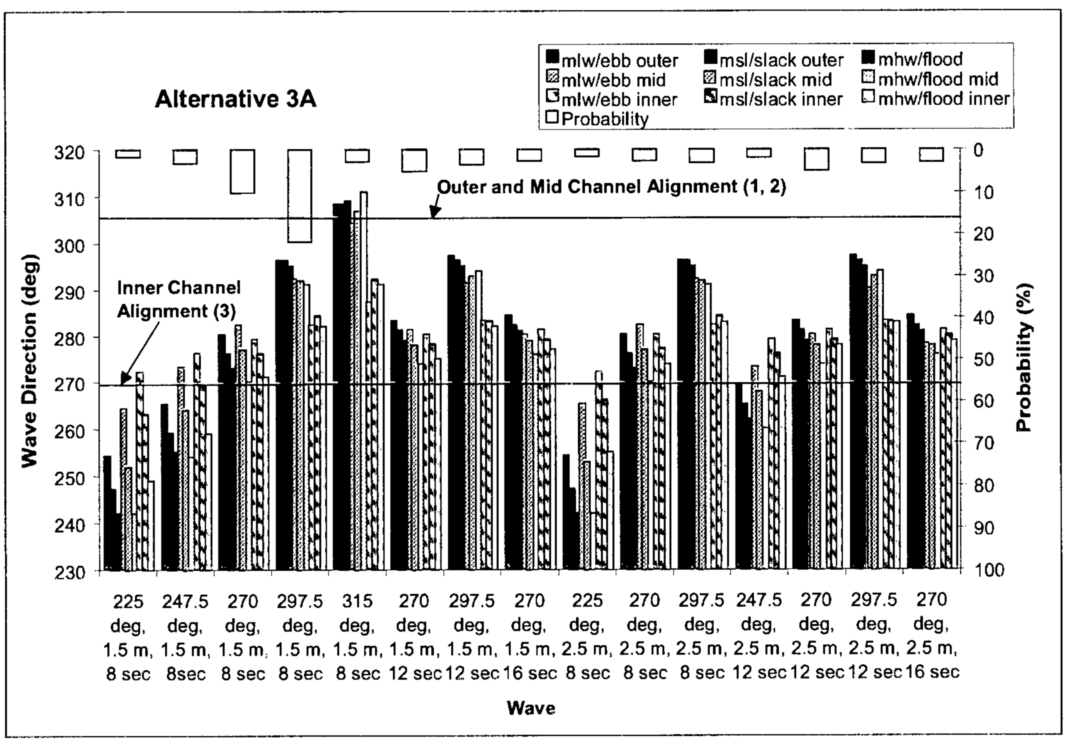


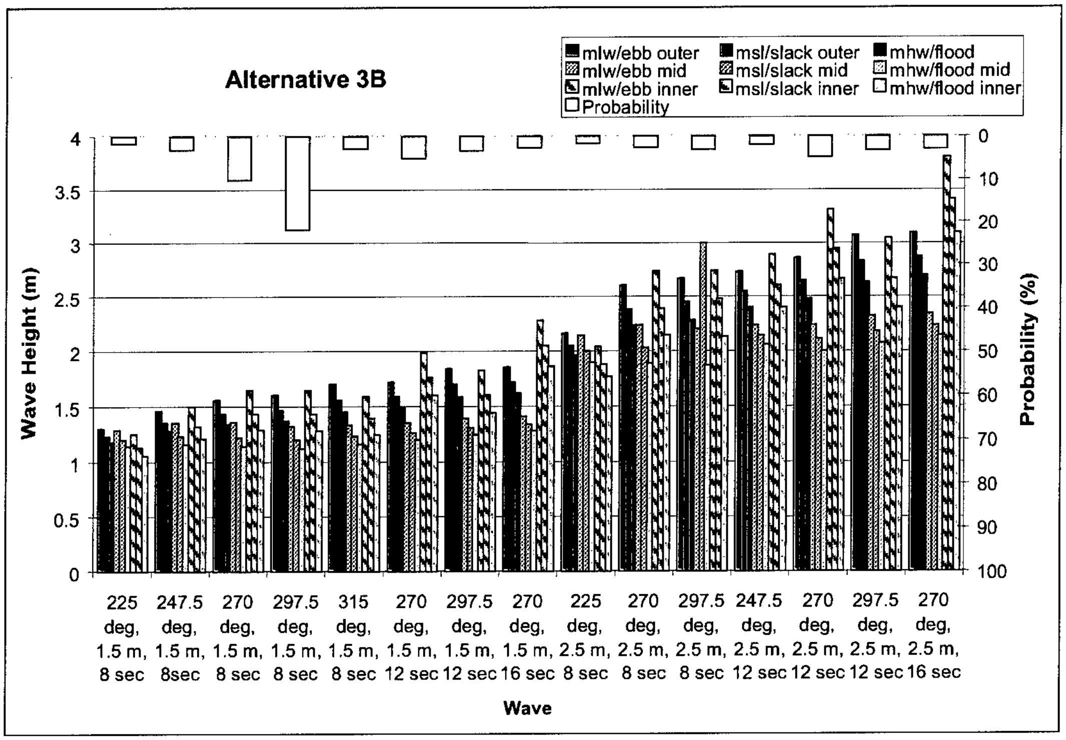
Alternative 1 (Existing)
mlw/ebb outer
mlw/ebb mid
mlw/ebb inner
Probability
msl/slack outer
msl/slack mid
msl/slack inner
mhw/flood
mhw/flood mid
mhw/flood inner
Wave Height (m)
Probability (%)
225 deg, 1.5 m, 8 sec
247.5 deg, 1.5 m, 8sec
270 deg, 1.5 m 8, sec
297.5 deg, 1.5 m, 8 sec
315 deg, 1.5 m, 8 sec
270 deg, 1.5 m, 12 sec
297.5 deg, 1.5 m, 12 sec
270 deg, 1.5 m, 16 sec
225 deg, 2.5 m, 8 sec
270 deg, 2.5 m, 8 sec
297.5 deg, 2.5 m, 8 sec
247.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 12 sec
297.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 16 sec
Wave

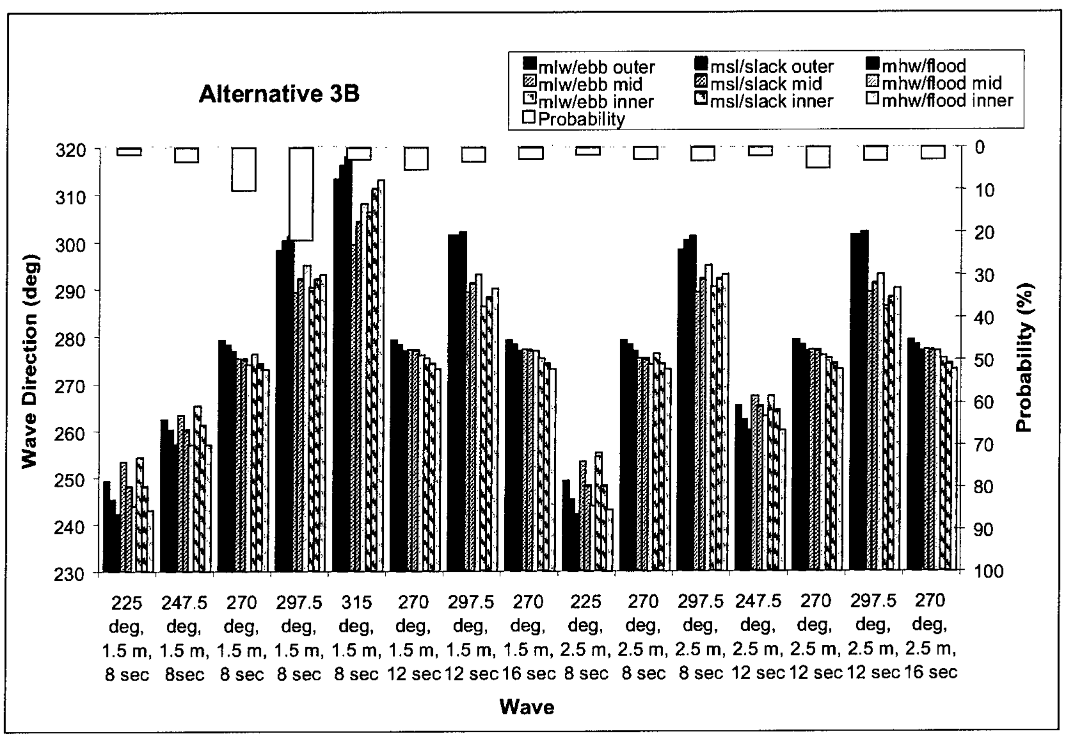


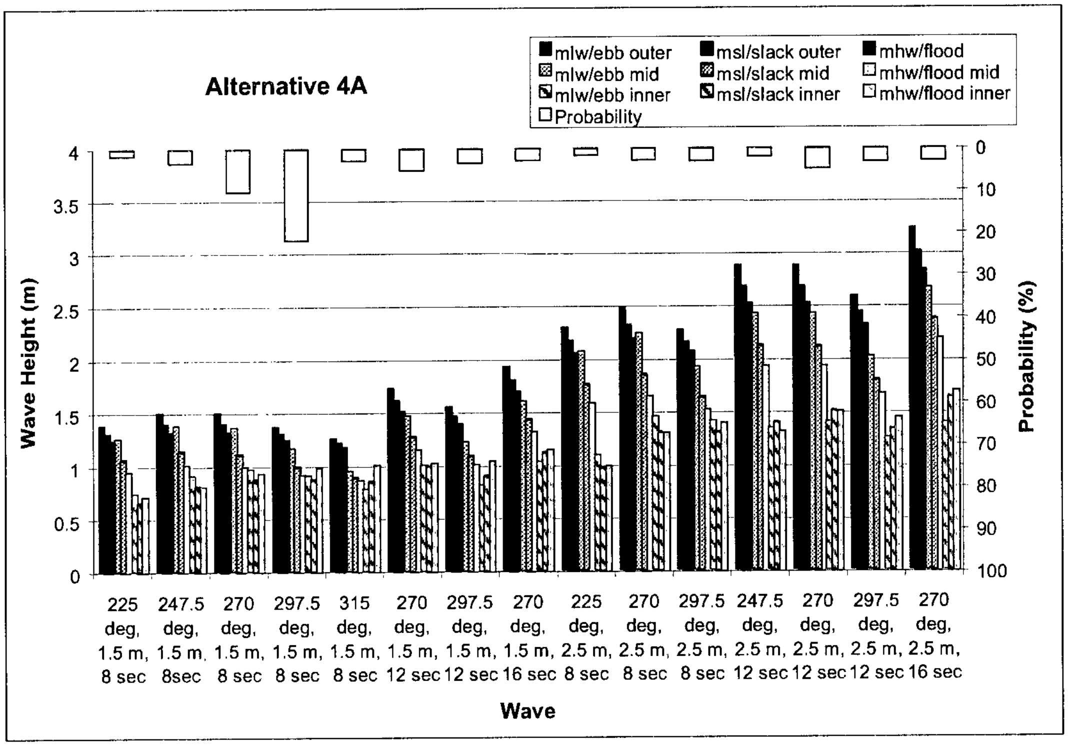
Alternative 1 (Existing)
mlw/ebb outer
mlw/ebb mid
mlw/ebb inner
Probability
msl/slack outer
msl/slack mid
msl/slack inner
mhw/flood
mhw/flood mid
mhw/flood inner
Inner and Mid Channel Alignment (1, 2)
Inner Channel Alignment (3)
Wave Direction (deg)
Probability (%)
225 deg, 1.5 m, 8 sec
247.5 deg, 1.5 m, 8sec
270 deg, 1.5 m 8, sec
297.5 deg, 1.5 m, 8 sec
315 deg, 1.5 m, 8 sec
270 deg, 1.5 m, 12 sec
297.5 deg, 1.5 m, 12 sec
270 deg, 1.5 m, 16 sec
225 deg, 2.5 m, 8 sec
270 deg, 2.5 m, 8 sec
297.5 deg, 2.5 m, 8 sec
247.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 12 sec
297.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 16 sec
Wave

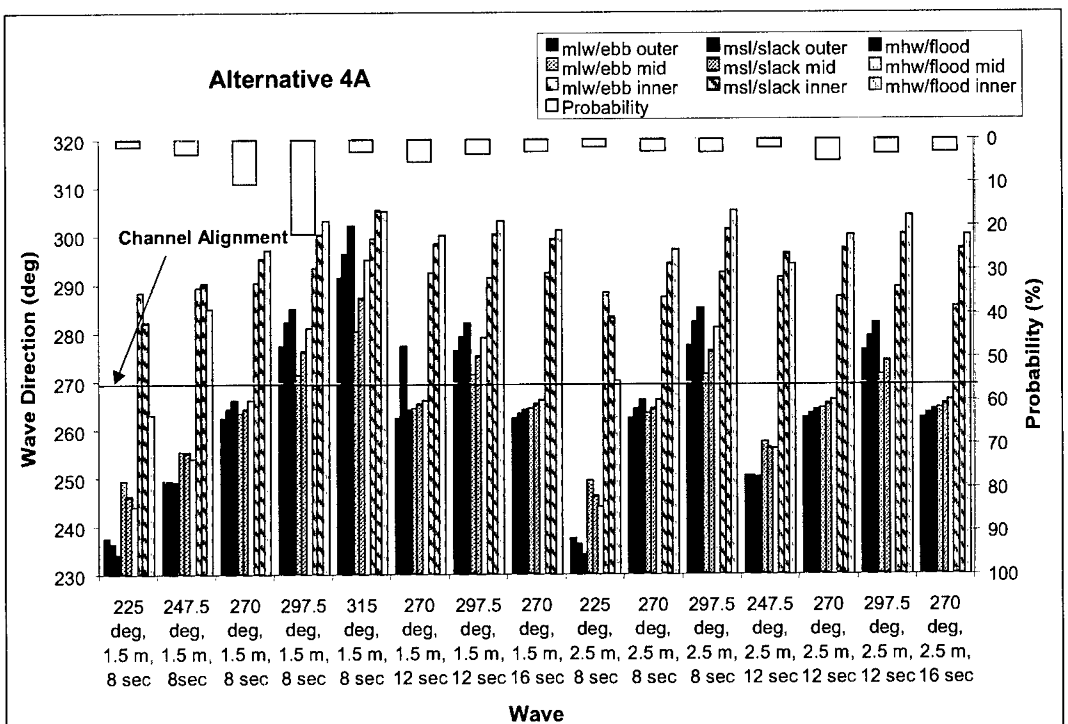

Alternative 3A
mlw/ebb outer
mhw/flood
msl/slack mid
mlw/ebb inner
mhw/flood inner
msl/slack outer
mlw/ebb mid
mhw/flood mid
msl/slack inner
Probability
Wave Height (m)
Probability (%)
225 deg, 1.5 m, 8 sec
247.5 deg, 1.5 m, 8sec
270 deg, 1.5 m, 8 sec
297.5 deg, 1.5 m, 8 sec
315 deg, 1.5 m, 8 sec
270 deg, 1.5 m, 12 sec
297.5 deg, 1.5 m, 12 sec
270 deg, 1.5 m, 16 sec
225 deg, 2.5 m, 8 sec
270 deg, 2.5 m, 8 sec
297.5 deg, 2.5 m, 8 sec
247.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 12 sec
297.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 16 sec
Wave

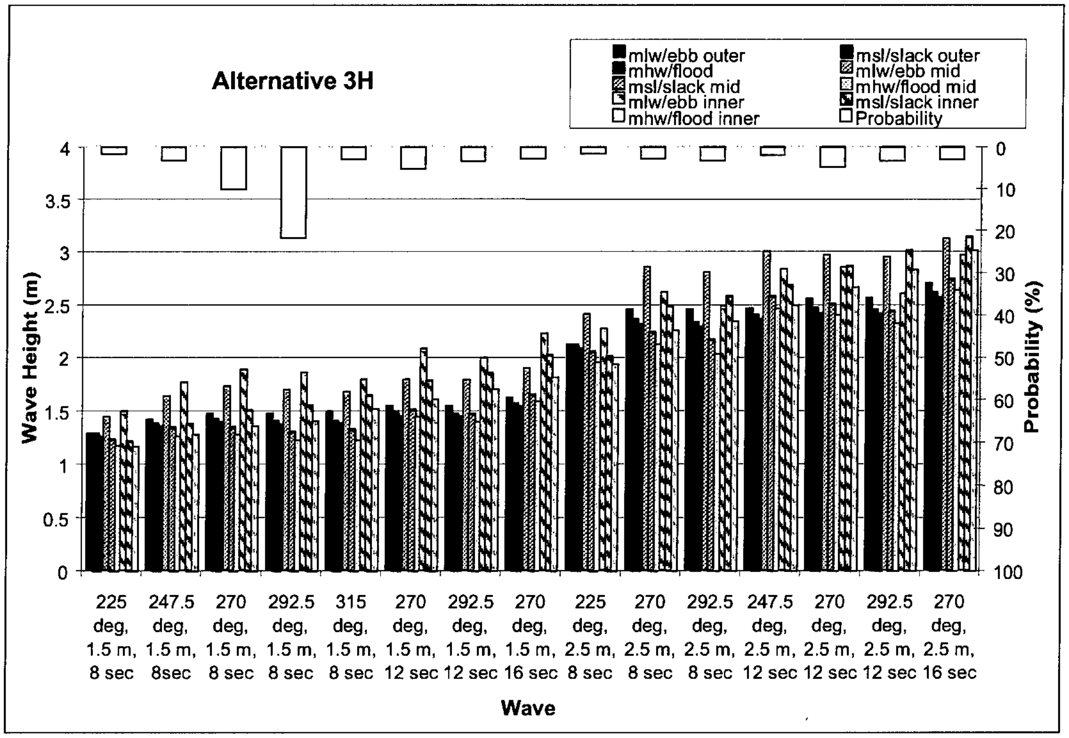

Alternative 3A
mlw/ebb outer
msl/slack outer
mhw/flood
mlw/ebb mid
msl/slack mid
mhw/flood mid
mlw/ebb inner
msl/slack inner
mhw/flood inner
Probability
Outer and Mid Channel Alignment (1, 2)
Inner Channel Alignment (3)
Wave Direction (deg)
Probability (%)
225 deg, 1.5 m, 8 sec
247.5 deg, 1.5 m, 8sec
270 deg, 1.5 m, 8 sec
297.5 deg, 1.5 m, 8 sec
315 deg, 1.5 m, 8 sec
270 deg, 1.5 m, 12 sec
297.5 deg, 1.5 m, 12 sec
270 deg, 1.5 m, 16 sec
225 deg, 2.5 m, 8 sec
270 deg, 2.5 m, 8 sec
297.5 deg, 2.5 m, 8 sec
247.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 12 sec
297.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 16 sec
Wave


Alternative 3B
mlw/ebb outer
mlw/ebb mid
mlw/ebb inner
msl/slack outer
msl/slack mid
msl/slack inner
mhw/flood
mhw/flood mid
mhw/flood inner
Probability
Wave Height (m)
Probability (%)
0
0.5
1
1.5
2
2.5
3
3.5
4
0
10
20
30
40
50
60
70
80
90
100
225 deg, 1.5 m, 8 sec
247.5 deg, 1.5 m, 8sec
270 deg, 1.5 m, 8 sec
297.5 deg, 1.5 m, 8 sec
315 deg, 1.5 m, 8 sec
270 deg, 1.5 m, 12 sec
297.5 deg, 1.5 m, 12 sec
270 deg, 1.5 m, 16 sec
225 deg, 2.5 m, 8 sec
270 deg, 2.5 m, 8 sec
297.5 deg, 2.5 m, 8 sec
247.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 12 sec
297.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 16 sec
Wave

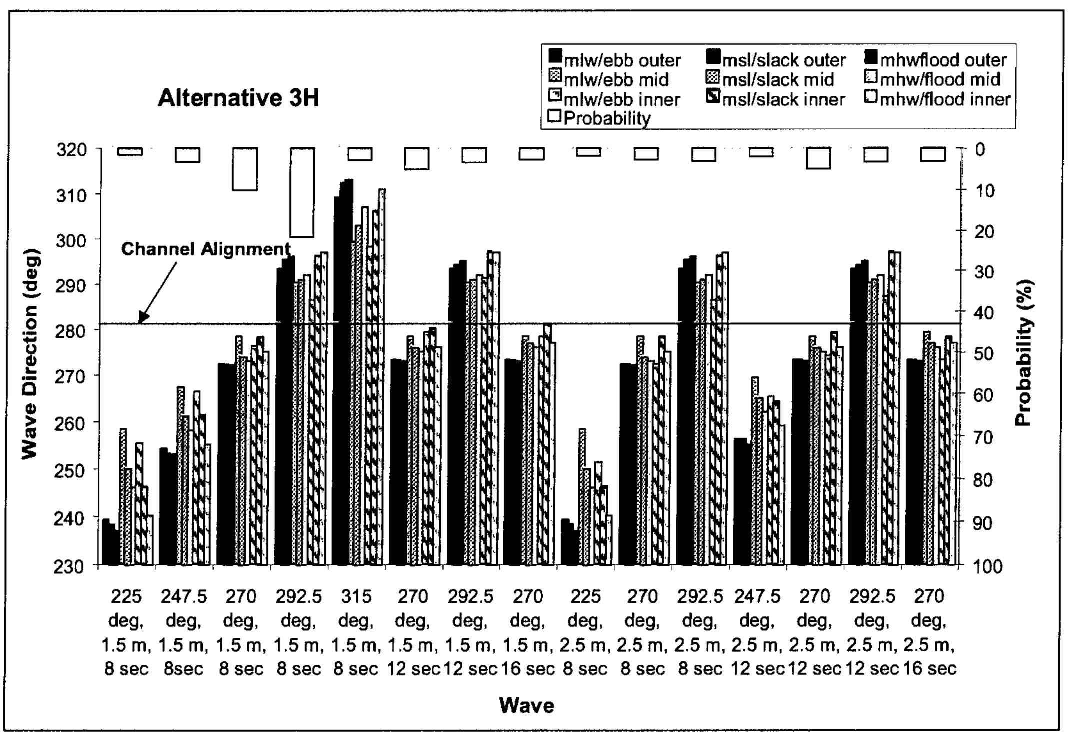

Alternative 3B
mlw/ebb outer
mlw/ebb mid
mlw/ebb inner
msl/slack outer
msl/slack mid
msl/slack inner
mhw/flood
mhw/flood mid
mhw/flood inner
Probability
Wave Direction (deg)
Probability (%)
230
240
250
260
270
280
290
300
310
320
0
10
20
30
40
50
60
70
80
90
100
225 deg, 1.5 m, 8 sec
247.5 deg, 1.5 m, 8sec
270 deg, 1.5 m, 8 sec
297.5 deg, 1.5 m, 8 sec
315 deg, 1.5 m, 8 sec
270 deg, 1.5 m, 12 sec
297.5 deg, 1.5 m, 12 sec
270 deg, 1.5 m, 16 sec
225 deg, 2.5 m, 8 sec
270 deg, 2.5 m, 8 sec
297.5 deg, 2.5 m, 8 sec
247.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 12 sec
297.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 16 sec
Wave


Alternative 4A
mlw/ebb outer
mlw/ebb mid
mlw/ebb inner
msl/slack outer
msl/slack mid
msl/slack inner
mhw/flood
mhw/flood mid
mhw/flood inner
Probability
Wave Height (m)
Probability (%)
225 deg, 1.5 m, 8 sec
247.5 deg, 1.5 m, 8sec
270 deg, 1.5 m, 8 sec
297.5 deg, 1.5 m, 8 sec
315 deg, 1.5 m, 8 sec
270 deg, 1.5 m, 12 sec
297.5 deg, 1.5 m, 12 sec
270 deg, 1.5 m, 16 sec
225 deg, 2.5 m, 8 sec
270 deg, 2.5 m, 8 sec
297.5 deg, 2.5 m, 8 sec
247.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 12 sec
297.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 16 sec
Wave


Alternative 4A
mlw/ebb outer
mlw/ebb mid
mlw/ebb inner
msl/slack outer
msl/slack mid
msl/slack inner
mhw/flood
mhw/flood mid
mhw/flood inner
Probability
Channel Alignment
Wave Direction (deg)
Probability (%)
225 deg, 1.5 m, 8 sec
247.5 deg, 1.5 m, 8sec
270 deg, 1.5 m, 8 sec
297.5 deg, 1.5 m, 8 sec
315 deg, 1.5 m, 8 sec
270 deg, 1.5 m, 12 sec
297.5 deg, 1.5 m, 12 sec
270 deg, 1.5 m, 16 sec
225 deg, 2.5 m, 8 sec
270 deg, 2.5 m, 8 sec
297.5 deg, 2.5 m, 8 sec
247.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 12 sec
297.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 16 sec
Wave



Alternative 3H
mlw/ebb outer
msl/slack outer
mhw/flood
mlw/ebb mid
msl/slack mid
mhw/flood mid
mlw/ebb inner
msl/slack inner
mhw/flood inner
Probability
Wave Height (m)
Probability (%)
225 deg, 1.5 m, 8 sec
247.5 deg, 1.5 m, 8sec
270 deg, 1.5 m, 8 sec
292.5 deg, 1.5 m, 8 sec
315 deg, 1.5 m, 8 sec
270 deg, 1.5 m, 12 sec
292.5 deg, 1.5 m, 12 sec
270 deg, 1.5 m, 16 sec
225 deg, 2.5 m, 8 sec
270 deg, 2.5 m, 8 sec
292.5 deg, 2.5 m, 8 sec
247.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 12 sec
292.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 16 sec
Wave



Alternative 3H
mlw/ebb outer
msl/slack outer
mhwflood outer
mlw/ebb mid
msl/slack mid
mhw/flood mid
mlw/ebb inner
msl/slack inner
mhw/flood inner
Probability
Channel Alignment
Wave Direction (deg)
Probability (%)
225 deg, 1.5 m, 8 sec
247.5 deg, 1.5 m, 8sec
270 deg, 1.5 m, 8 sec
292.5 deg, 1.5 m, 8 sec
315 deg, 1.5 m, 8 sec
270 deg, 1.5 m, 12 sec
292.5 deg, 1.5 m, 12 sec
270 deg, 1.5 m, 16 sec
225 deg, 2.5 m, 8 sec
270 deg, 2.5 m, 8 sec
292.5 deg, 2.5 m, 8 sec
247.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 12 sec
292.5 deg, 2.5 m, 12 sec
270 deg, 2.5 m, 16 sec
Wave

Appendix E Wave Climate Intercomparisons[1]

Wave-climate statistics from the Grays Harbor buoy are accessed in this report to describe the wave climate incident at Willapa Bay (Chapter 5). Wave data from the Long Beach slope array and the Columbia River bar buoy were also analyzed to collaborate climate information obtained from the Grays Harbor buoy. Differences were found between the Grays Harbor and Long Beach wave directional distributions. The purpose of this appendix is to discuss these differences and document why the Grays Harbor data were selected to provide offshore boundary conditions for wave modeling conducted in this project.

The Grays Harbor, Long Beach, and Columbia River bar gauges are described in Chapter 5. For review, the Grays Harbor buoy is located northwest of Willapa Bay (closest gauge to the entrance) in a water depth of 40 m. The Long Beach slope array was southwest of the entrance in a water depth of 10 m. The Columbia River bar buoy is still further southwest in a water depth of 128 m. Table E-1 shows the percent occurrence of wave directions for the Grays Harbor buoy, Long Beach slope array, and the Columbia River bar buoy. In this table, each month is given the appropriate weight (based on the number of days in the month), so large gaps in the data sets do not bias the statistics toward a particular month or season. Also, Grays Harbor data with sampling intervals of 1 hr or less were decimated to 3-hr intervals for consistency. The percent coverage of data for each month is given in Table E-2. The Grays Harbor and Columbia River buoys have similar directional distributions, with the Columbia River buoy recording slightly more waves from the northwest and fewer from the southwest.

The Long Beach array gives a much different distribution with more waves from the west (270-deg band), more from the southwest (247.5-deg band), and fewer from the northwest (292.5-deg band) (Table E-1). These directional differences between the gauges could reverse the direction of estimated longshore sediment transport and change the magnitude of transport. Thus, some explanation of the differences is required to select the most appropriate climate for furnishing offshore boundary conditions to the wave transformation model.

[1] Written by Dr. Jane McKee Smith, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS.

Table E-1 Wave Direction Distributions			
Direction Deg	**Grays Harbor Buoy % occurrence**	**Long Beach Array % occurrence**	**Columbia River Bar Buoy % occurrence**
180.0	0.6	1.8	1.0
202.5	3.8	2.7	4.6
225.0	9.4	8.9	7.6
247.5	12.1	31.4	8.2
270.0	37.8	45.7	33.0
292.5	30.5	8.3	32.0
315.0	5.3	0.4	12.0
337.5	0.1	0.0	1.1
360.0	0.0	0.0	0.4

Table E-2 Percent Data Coverage by Month			
Month	**Grays Harbor Buoy % coverage**	**Long Beach Slope Array % coverage**	**Columbia River Bar Buoy % coverage**
Jan	8.7	8.3	9.8
Feb	8.4	7.7	3.1
Mar	8.6	8.4	3.4
Apr	5.9	8.1	3.3
May	7.4	8.0	9.6
Jun	7.7	7.0	10.0
Jul	9.4	5.7	10.9
Aug	8.9	8.3	11.0
Sep	9.7	10.0	10.3
Oct	10.1	10.8	10.4
Nov	7.3	9.0	10.0
Dec	7.9	8.6	8.2

Figure E-1 compares a time-history of zero-moment wave height measured at the Grays Harbor and Long Beach gauges for March 1995 (one of the few coincident months of data). The heights agree well for this month. For some events, the Long Beach heights are higher, probably produced by local wave shoaling in the shallower water. For the largest wave height in the month, the Grays Harbor buoy wave height is higher, probably because of depth-limited breaking at the Long Beach gauge. Figure E-2 compares significant wave direction for the same month. The directions for the two gauges show similar trends, but there are significant differences. First, the Grays Harbor directions

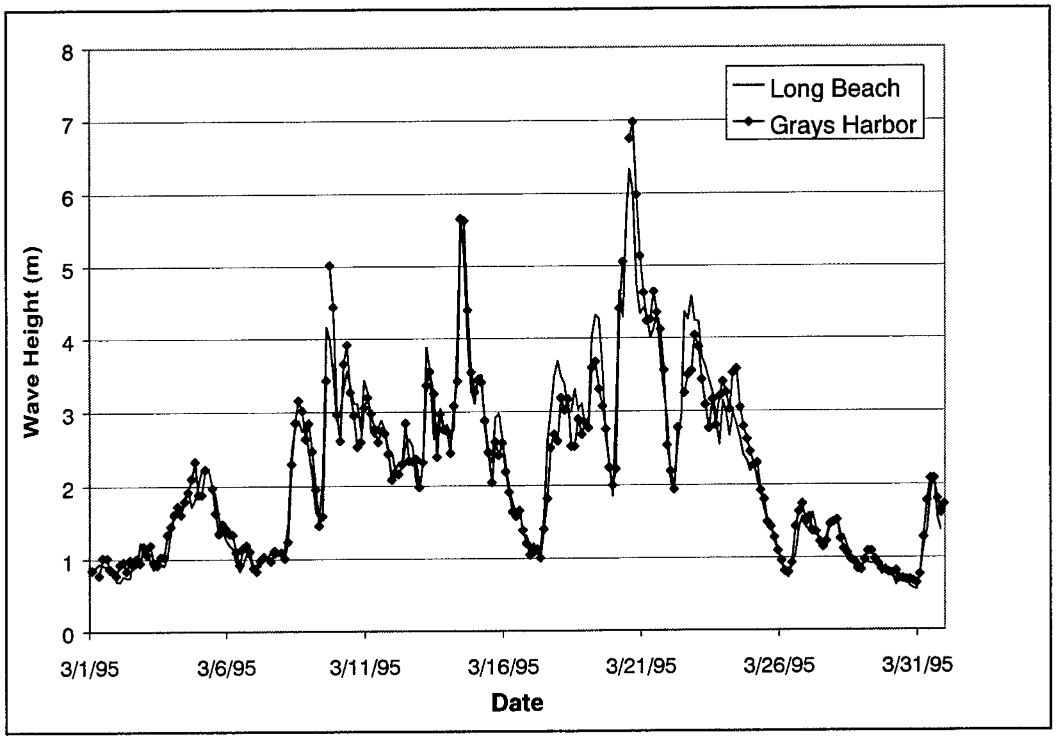


Figure E-1. March 1995 wave heights for Long Beach slope array and Grays Harbor buoy

are generally more oblique (further from 270 deg). Reduced obliqueness at the Long Beach gauge is caused by more refraction (wave aligning with the normal to shore, 270 deg). Second, the Long Beach directions oscillate 5 to 25 deg with the tide (most obvious in the last third of the record), caused by refraction over the tide-varying water depth. The tidal variation in direction is strongest at Long Beach because the tide range is a large fraction of the water depth (on the order of one-third the depth).

Figure E-3 shows a scatter plot of measured directions for March 1995 for the Grays Harbor and Long Beach gauges. This shows again the reduced spread in direction at the Long Beach gauge caused by refraction. The mean wave direction for March 1995 was 256.8 deg at the Grays Harbor gauge and 253.2 deg at the Long Beach gauge. Figures E-2 and E-3 support the increased occurrence of waves in the 270-deg band at Long Beach (45.3 percent at Long Beach versus 37.8 at Grays Harbor), but they do not explain why the Long Beach array distribution is skewed to the south and that of the buoy is skewed to the north. The local bottom contour and shoreline orientation at both Grays Harbor and Long Beach are approximately north-south. Thus, local refraction does not explain the difference in distribution skewness. To better understand the difference in directional distribution, the monthly variation in direction is investigated in the next paragraph.

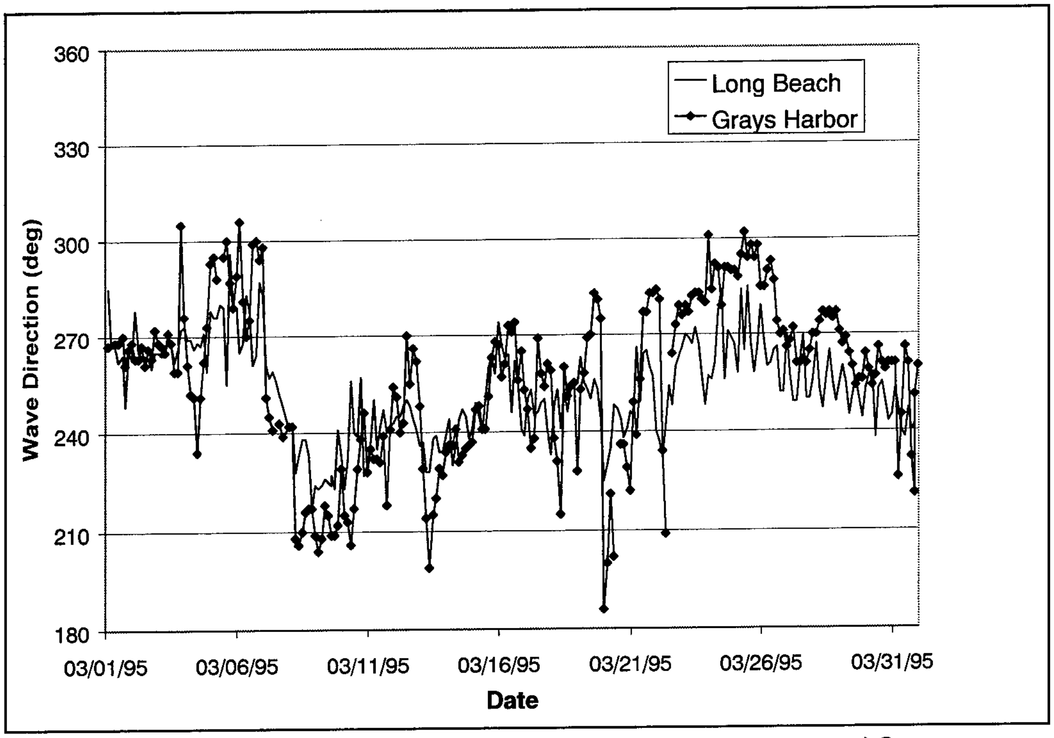


Figure E-2. March 1995 wave directions for Long Beach slope array and Grays Harbor buoy

The monthly-averaged wave directions for Grays Harbor and Long Beach are given in Figures E-4 and E-5, respectively. Each year is plotted separately to show year-to-year variability and anomalies in the data. The Grays Harbor buoy data exhibit a strong trend of southwesterly mean direction from November to April and northwesterly direction from May to October, with a total range of 240 to 300 deg. The Long Beach gauge data show much less seasonal variation in mean direction (range of 240 to 280 deg). In May through July, the monthly mean wave directions at Long Beach are 10 to 20 deg less than at Grays Harbor. During the other months, the mean directions are similar at the two gauges. Figure E-6 shows coincident months of mean wave directions for Grays Harbor and Columbia River (the Long Beach gauge stopped operating by the time the Columbia River buoy became directional). This figure indicates good seasonal agreement in wave direction between Grays Harbor and Columbia River. This agreement supports the accuracy of the Grays Harbor measurements and brings into question the measured directions by the Long Beach gauge.

Because of the good direction agreement between Grays Harbor and Columbia River buoys, which bracket the Willapa entrance and the Long Beach gauge position, it is concluded that the Grays Harbor gauge is most representative of the wave climate at Willapa Bay. The difference in summer wave angles at the Long Beach gauge may be related to limitations of the bottom-mounted pressure gauge in resolving wave direction for the shorter-period summer waves in 10-m water depth. In addition to the apparent problems in resolving the direction of summer (shorter period) waves at Long Beach, the gauge has a compressed directional variation because of refraction, increased variance in direction introduced by the tide, and depth limitations for maximum

wave height. These factors make the Grays Harbor buoy the appropriate source of wave climate information for the Willapa entrance.

People with local experience navigating the Willapa entrance have stated that waves from the southwest are significant and suggested that they may be underrepresented in the Grays Harbor climate. This perception of under-representation may owe to the fact that although the waves south of west comprise only 26 percent of the distribution, these waves include the large winter storm waves that can limit navigation and are the most visible.

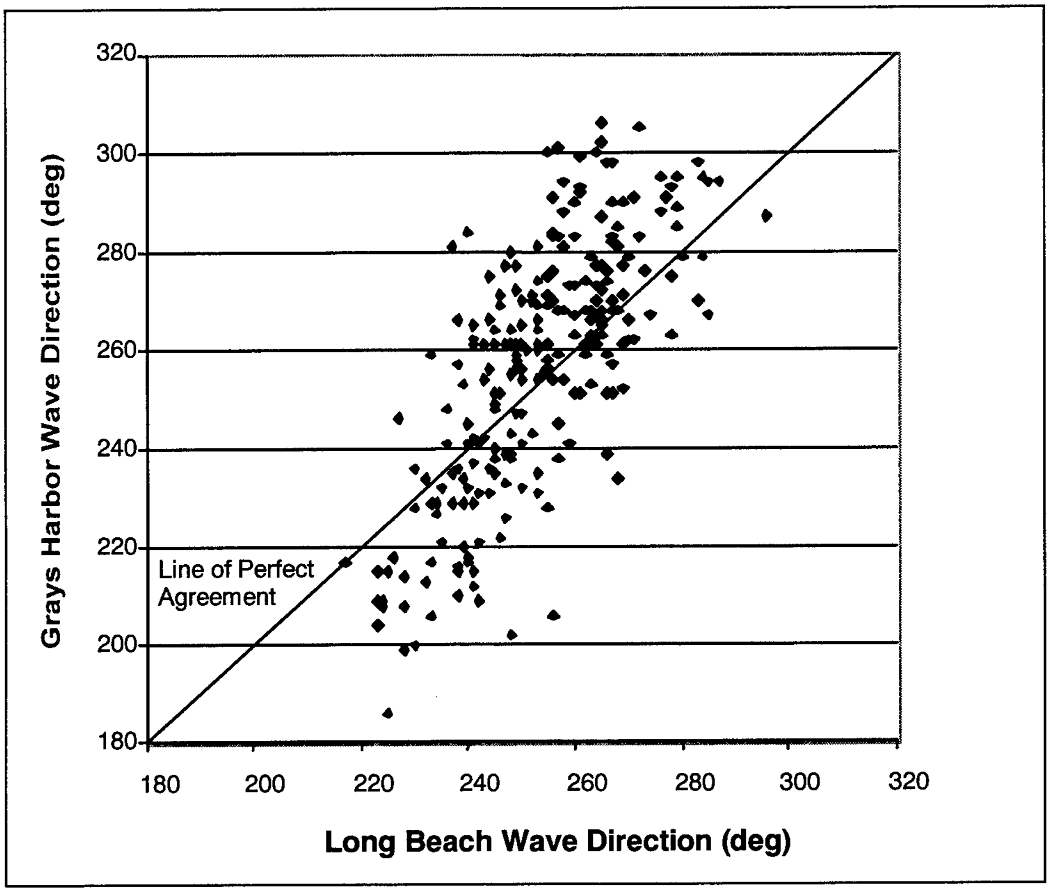


Figure E-3. Scatter plot of Grays Harbor and Long Beach wave directions for March 1995

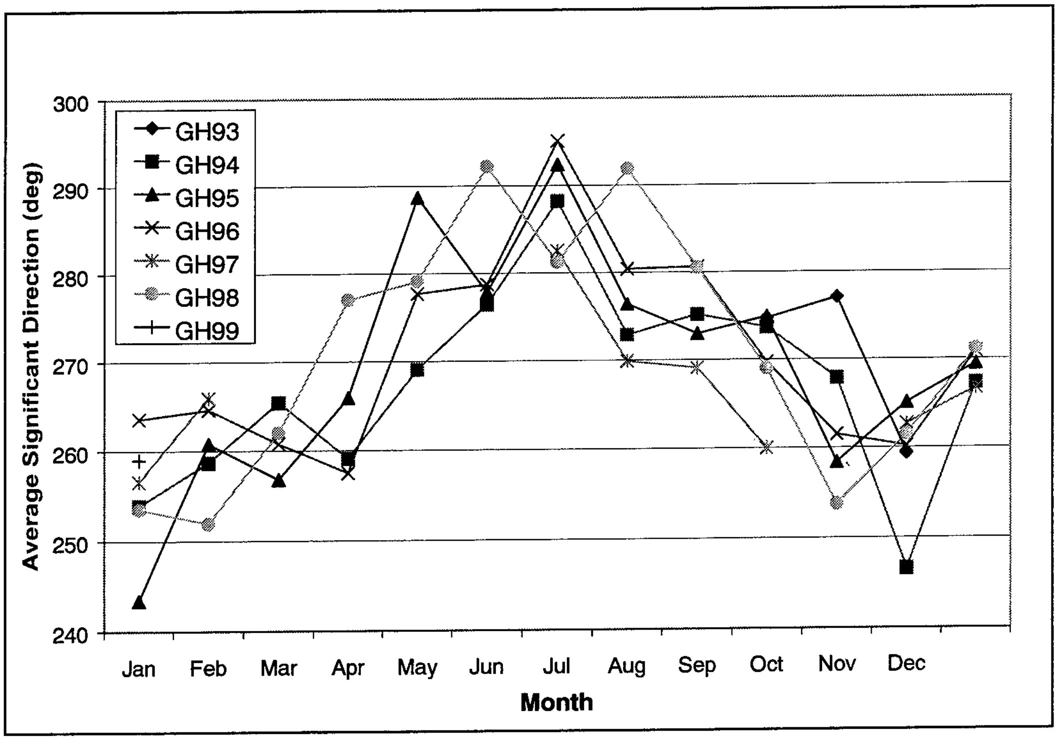


Figure E-4. Monthly average wave directions for Grays Harbor buoy

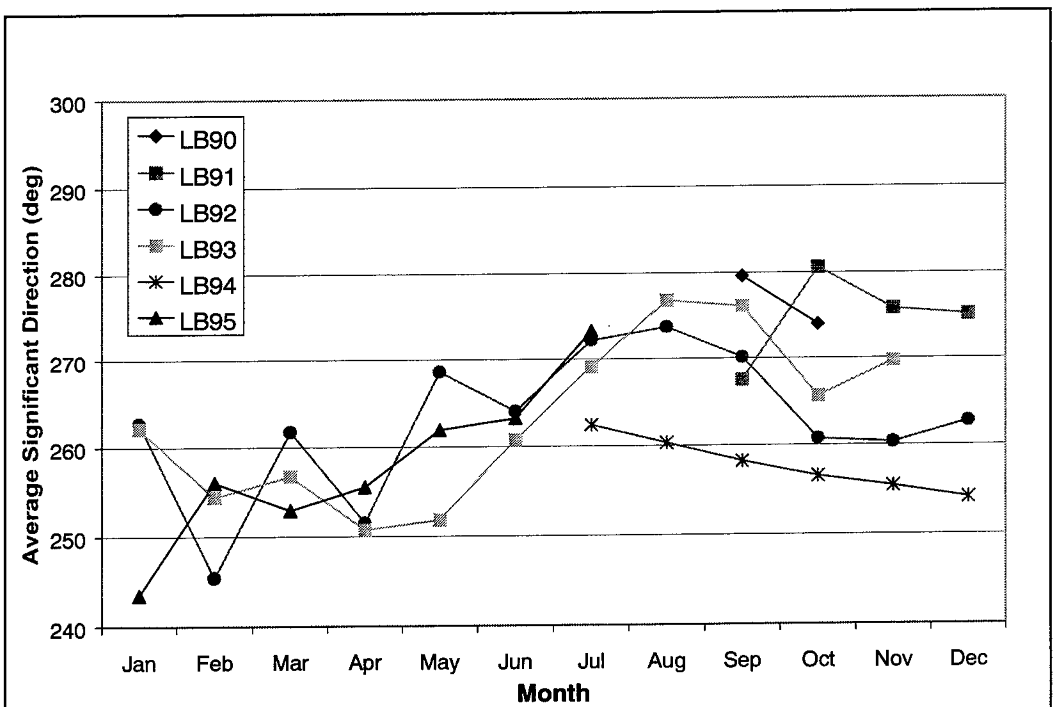


Figure E-5. Monthly average wave directions for Long Beach slope array

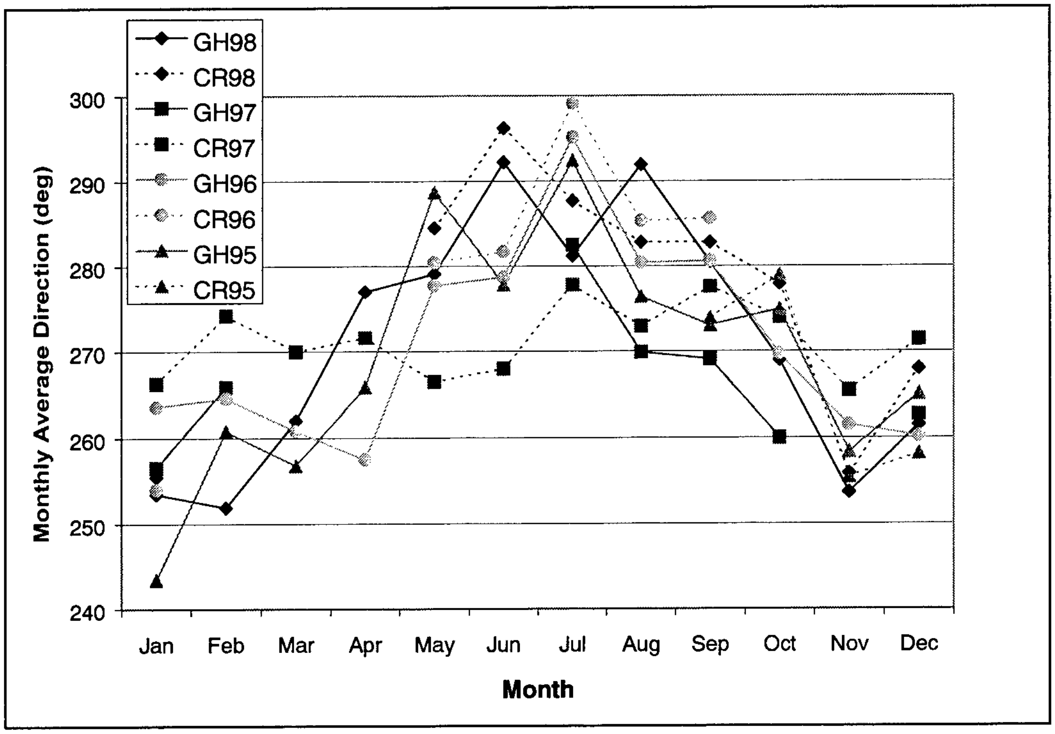


Figure E-6. Monthly average wave directions for coincident Grays Harbor buoy and Columbia River bar buoy measurements

Appendix F Comparison of Buoy and Pressure Gauge Measurements[1]

Wave measurements in Willapa Bay at Stations 2 and 3 were made with bottom-mounted pressure gauges in water depths of approximately 10 m (Chapter 4). The maximum current magnitudes at Stations 2 and 3 were approximately 1.5 m/sec on ebb and 1.0 m/sec on flood. The bottom-mounted pressure gauges were used to estimate the wave height on the surface by correcting for the attenuation of the wave pressure between the surface and the bottom. This correction factor is a function of water depth and tidal current. Examination of the surface-corrected pressure measurements showed spectral energy densities that increased exponentially at frequencies greater than about 0.2 to 0.3 Hz. This divergence of the spectrum occurred at lower frequencies during flood currents (0.2 to 0.25 Hz) and at higher frequencies during ebb currents (0.25 to 0.3 Hz). Wave heights calculated from the surface-corrected pressure spectra are dependent on the choice of the spectral cutoff (the higher the cutoff, the larger the wave height).

To quantify the appropriate cutoff to be specified in the wave analysis, a wave buoy and pressure gauge were installed at Station 2 for two short-term deployments. The wave buoy follows the water surface and thus does not require application of a correction factor as does the pressure gauge. In this appendix, the spectra from the surface-following buoy and the bottom-mounted pressure gauge are compared to select the most appropriate spectral cutoff. The purpose of this appendix is to describe the comparison of the buoy and pressure gauge data and document the cutoff used in analysis of the pressure data for Stations 2 and 3.

[1] Written by Dr. Jane McKee Smith, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS, Messrs. Keith Kurrus and Charles E. Abbott, Evans-Hamilton, Inc., and Mr. Scott Fenical, Pacific International Engineering^{PLLC}.

Gauge Deployment

The wave buoy deployed was a 0.9-m diameter, nondirectional Waverider manufactured by Datawell B.V. The pressure gauge was a Paroscientific, Inc., pressure sensor, which was deployed on a tripod (approximately 1 m off the bottom) with a SonTek ADVOcean current meter to give directional characteristics and current magnitude and direction. The gauges were deployed on 14 May 1999. The tripod was recovered on 28 May 1999. Data quality checks in the field revealed the tripod fell on its side after approximately two days for unknown reasons. After the tripod fell, output from the ADVOcean pitch, roll, and compass sensors was invalid, producing corrupted current data. Therefore, the tripod was redeployed on 28 May to acquire additional measurements. Both the buoy and tripod were recovered on 9 June 1999, and data quality checks confirmed the tripod remained upright throughout the second deployment. These two deployments are denoted as Deployments 7 and 8, respectively (the previous deployments are documented in Chapter 4). The locations and depths of the gauges are given in Table F-1.

Table F-1
Gauge Deployment Times and Locations

Deployment	Waverider Buoy	Tripod Deployment 7	Tripod Deployment 8
Deployment date[1]	14 May 1999 13:55 GMT	14 May 1999 14:12 GMT	28 May 1999 21:21 GMT
Recovery date[1]	9 June 1999 23:00 GMT	28 May 1999 13:45 GMT	9 June 1999 22:45 GMT
Latitude	46° 41' 46.88547"	46° 41' 47.77641"	46° 41' 47.98735"
Longitude	124° 05' 57.94298"	124° 05' 55.3766"	124° 05' 55.70806"
Depth	9.3 m mllw[2]	9.1 m mllw	9.1 m mllw

[1]Times given in Greenwich Mean Time (GMT).

[2]Mean lower low water.

Spectral Comparisons

Figures F-1 through F-3 show sample spectra from the buoy and pressure gauge. Figure F-1 shows a typical case where the pressure gauge spectrum diverges at around 0.3 Hz. This type of divergence is expected if a bottom-measured pressure signal is corrected to estimate the water-surface displacement. The pressure correction increases exponentially with frequency; at some ill-defined "high" frequency, the noise of the measurement system, multiplied by the pressure correction, far exceeds the wave signal. The spectrum should be truncated at a frequency below this point. The pressure correction is calculated as

$$\frac{1}{R_p^2} = \left\{ \frac{\cosh(kd)}{\cosh\left[k(z+d)\right]} \right\}^2 \qquad \text{(F-1)}$$

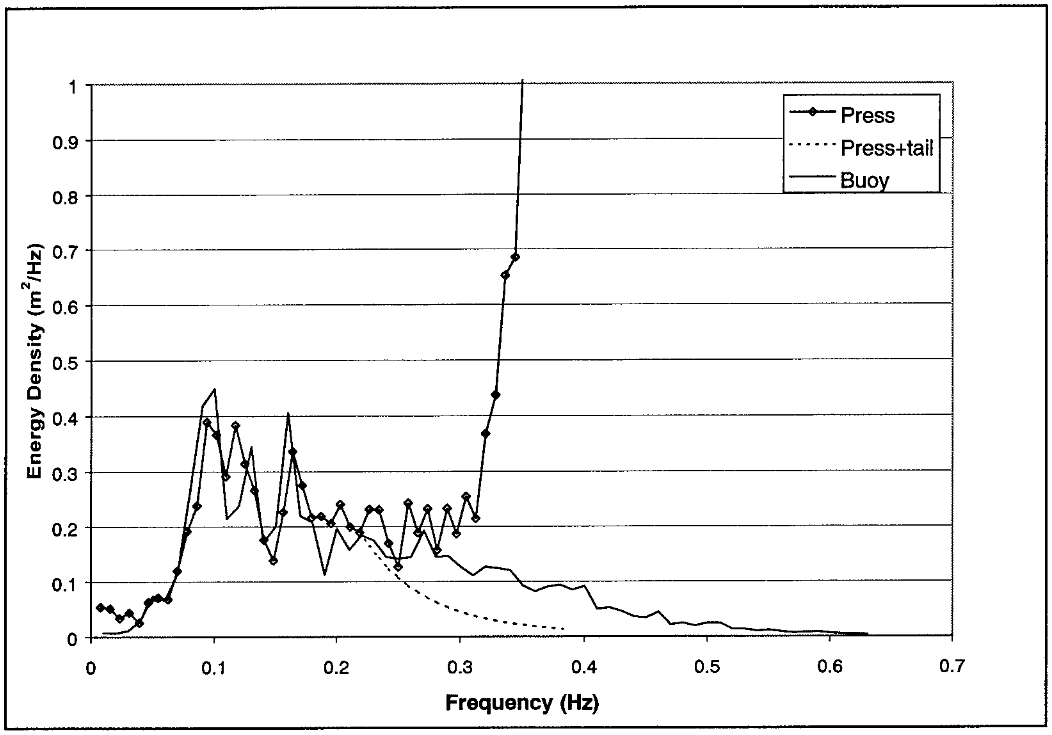


Figure F-1. Buoy and pressure gauge spectra for 8 June 1999 at 0:30 GMT

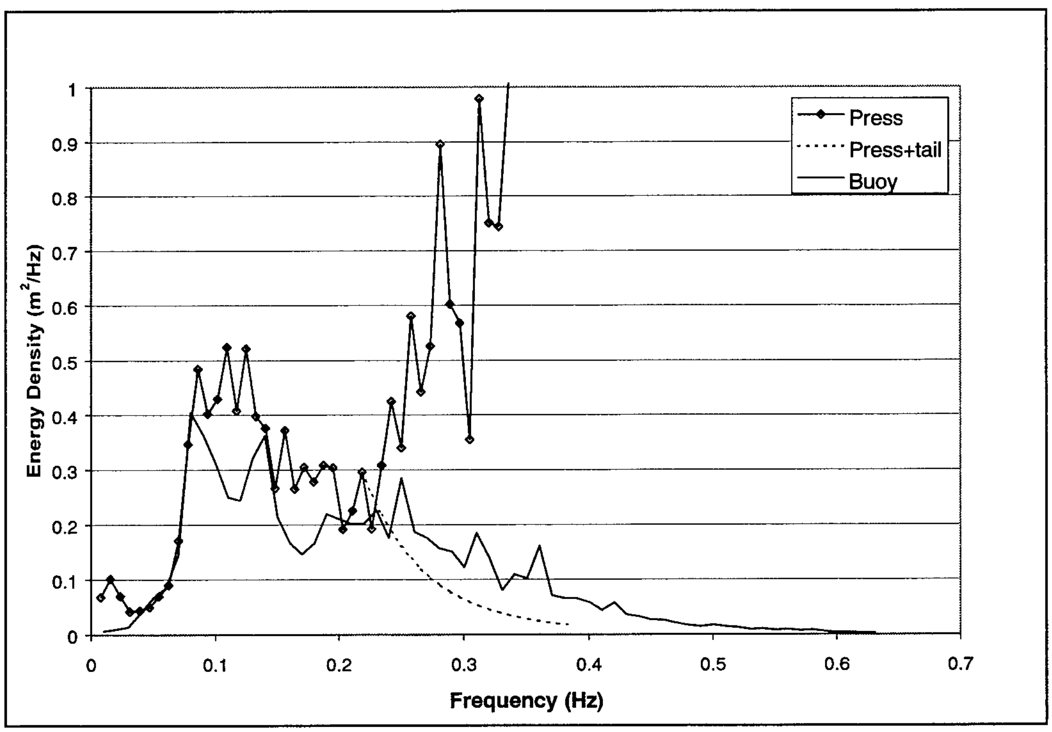


Figure F-2. Buoy and pressure gauge spectra for 7 June 1999 at 12:30 GMT

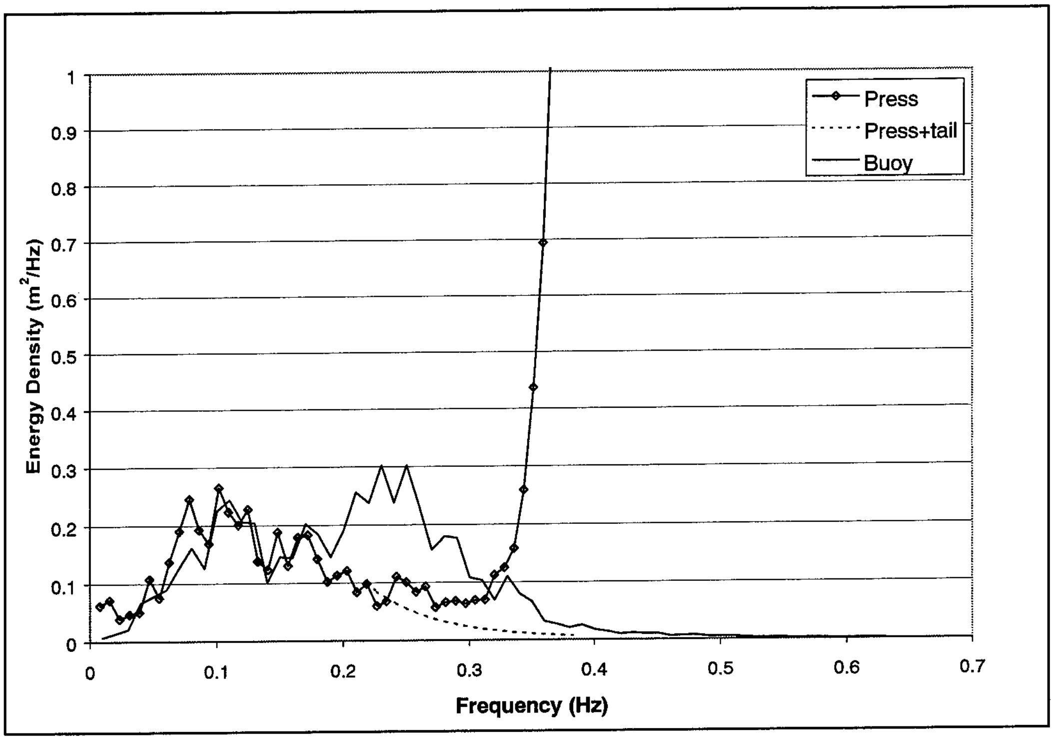


Figure F-3. Buoy and pressure gauge spectra for 7 June 1999 at 4:30 GMT

where

R_p = pressure-response function

k = wave number

d = water depth

z = elevation of the pressure gauge measured relative to the mean water surface

A typically applied cutoff is $1/(R_p)^2 = 100$. For the water depths at Station 2, this cutoff is 0.24 to 0.29 Hz (depending on tide elevation). Other environmental factors, such as the vertical variation in tidal currents, wave non-linearities, or wave breaking, may also affect the pressure signal, accelerating the divergence of the spectrum.

In Figure F-1, the agreement with the buoy spectrum is good up to 0.28 Hz. Figure F-2 shows a case where the pressure spectrum diverges at a lower frequency of 0.23 Hz. For this case, the wave height calculated from the pressure spectrum increases 35 percent if the cutoff is increased from 0.2 to 0.3 Hz. However, a cutoff of 0.2 Hz neglects higher frequency wave energy as measured by the buoy. Figure F-3 shows a midtide case where approximately three-fourths of the wave energy at Station 2 has been dissipated over the bar. This case is interesting because the buoy shows two peaks in the spectrum, one at 0.1 Hz (10-sec period) and one at 0.25 Hz (4-sec period). The higher peak is not represented in the pressure spectrum. The following conclusions can be drawn from the Deployment 8 buoy and pressure spectra:

a. The pressure spectra at Station 2 diverge in the frequency range of 0.2 to 0.3 Hz. The lower frequency divergences (0.2 to 0.25 Hz) occur while the current is flooding.

b. Even in cases where the spectrum does not diverge below 0.3 Hz, the pressure gauge is not capturing wave energy above about 0.2 Hz.

c. Applying a frequency cutoff of 0.2 Hz neglects significant wave energy as measured by the buoy.

Another way to examine the appropriate cutoff for the pressure gauge is to calculate the correlation between the buoy and pressure gauge spectra. Regions of high correlation indicate good agreement, and low correlation indicate poor agreement between the gauges. Figure F-4 shows the correlation for pressure gauge Deployment 8. The correlation coefficient (solid line) drops off rapidly above 0.22 Hz. Thus, a reasonable cutoff frequency is 0.22 Hz.

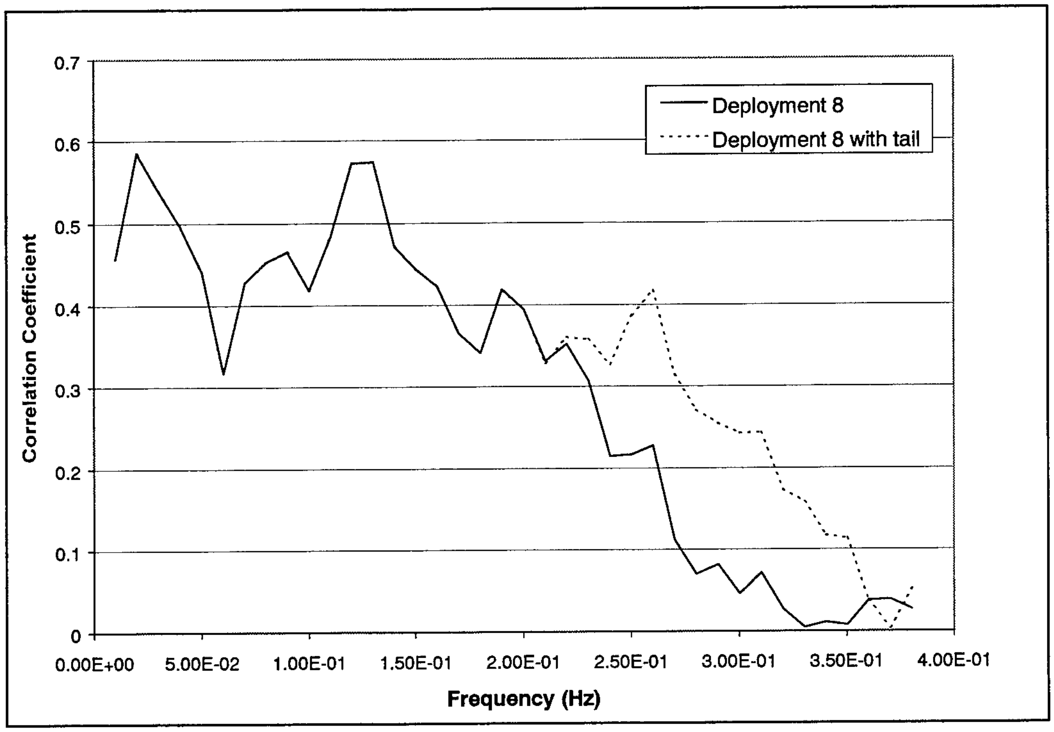


Figure F-4. Correlation between buoy and pressure gauge (Deployment 8)

Theoretical Spectral Tail

The previous section showed that the cutoff frequency should be approximately 0.22 Hz, but the spectral comparisons also show that significant energy (as measured by the buoy) may be neglected if the spectrum is truncated at 0.22 Hz. One method of estimating the truncated energy is to add a theoretical tail to the spectrum in the form given by Miller and Vincent (1990)

$$E(f) = \frac{\beta g^{-0.5} k^{-2.5}}{C_g} \tag{F-2}$$

where

$E(f)$ = spectral density at frequency f

β = constant (with units of m/sec)

g = gravitational acceleration

C_g = wave group celerity

A similar technique is employed by Wolf (1997). The value of β is estimated from the spectral density at the cutoff frequency. Examples of adding the spectral tail are shown as dashed lines in Figures F1-F3. The tail also improves the correlation coefficient at higher frequencies, as shown with the dashed line in Figure F-4. Adding the theoretical tail will not significantly improve the agreement for cases with peak periods shorter than 5 sec (frequency greater than 0.2 Hz), as seen in Figure F-3.

These shorter period waves may originate from three processes. First, these may be waves generated in Willapa Bay that are propagating out of the bay. Second, they may be harmonics generated as the waves shoal and are freed in the breaking process over the bar (Battjes and Beji 1992). Third, they may be wave components that shoal on the strong ebb currents. Whatever their origin, they are not reproduced in the pressure record analysis. Use of the near-bottom current measurement in the analysis, variation of the current over depth, and use of linear wave theory may contribute to the underestimate of shorter waves in the pressure gauge analysis. Although high-frequency wave energy is underestimated in some wave records, the following section shows that reasonable estimates of wave height and period are produced by applying a 0.22-Hz frequency cutoff with a theoretical tail.

Wave Height and Period Statistics

The choice of the spectral cutoff can also be evaluated based on wave height H_{mo} and peak period T_p comparisons between the buoy and pressure gauge. Figure F-5 compares the pressure gauge with a cutoff of 0.3 Hz, 0.22 Hz, and 0.22 Hz plus the theoretical tail with the buoy wave heights. The wave height with the 0.3-Hz cutoff generally overestimates the large wave heights at high tide. This is because the greater water depth at high tide causes the divergence region of the spectrum to shift down into the range below 0.3 Hz. At low tide, the 0.3-Hz cutoff underestimates the wave height because the pressure gauge is not capturing the higher frequency waves (e.g., Figure F-3). The wave height with the 0.22-Hz cutoff consistently underestimates the buoy wave heights by 0.05-0.15 m because it neglects any wave energy at frequencies above 0.22 Hz. The 0.22-Hz cutoff with the theoretical tail provides the best fit to the buoy wave heights, although it still underestimates the low-tide wave heights, similar to the other cutoff options.

Figure F-6 compares the pressure gauge with cutoffs of 0.3 Hz and 0.22 Hz with the buoy wave periods (adding the theoretical tail to the spectra produces the same peak periods as the 0.22-Hz cutoff). Both pressure gauge cutoffs give good results for wave periods greater than 5 sec. The 0.22-Hz cutoff cannot produce peak periods shorter than 4.5 sec. The 0.3-Hz cutoff can produce

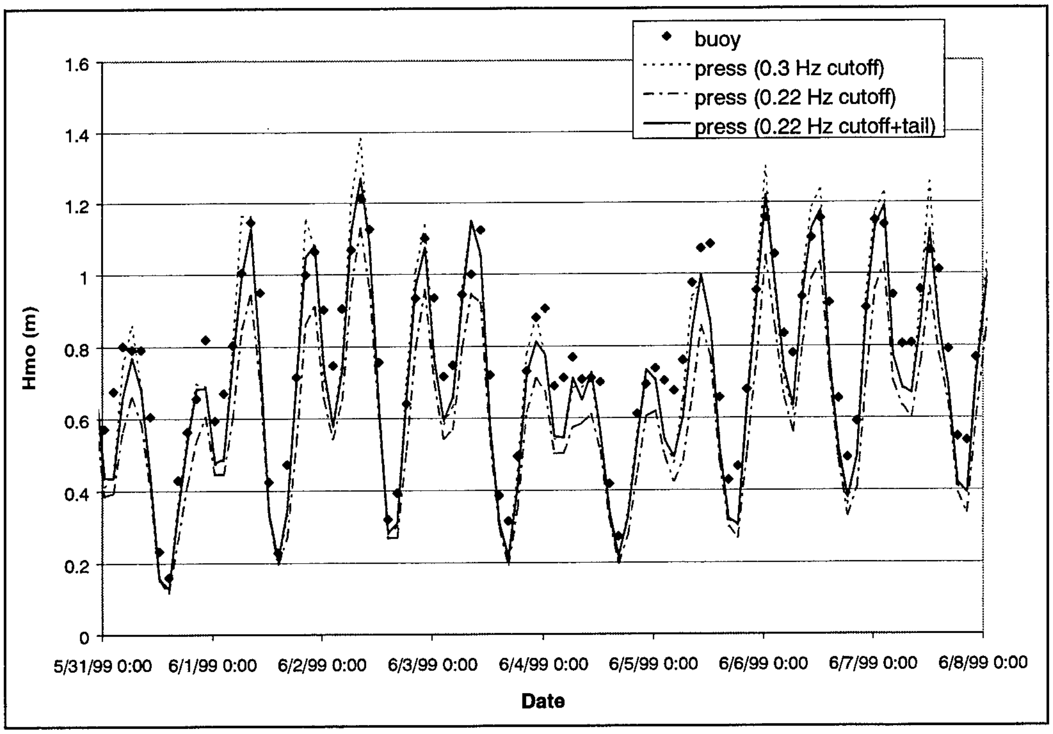


Figure F-5. Comparison of buoy and pressure gauge wave heights (Deployment 8)

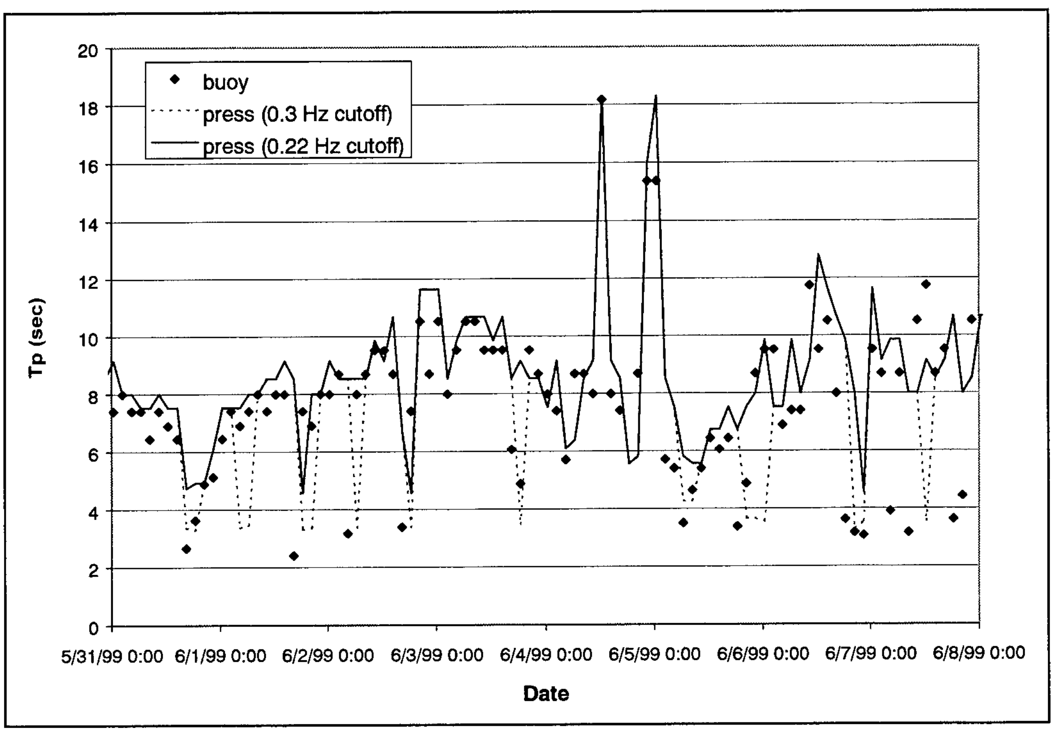


Figure F-6. Comparison of buoy and pressure gauge wave periods (Deployment 8)

periods as short as 3.3 sec, but these short-period estimates do not consistently match the buoy. Table F-2 lists root-mean-square (rms) errors in wave height and period for comparisons of the buoy and pressure gauge for Deployments 7 and 8. The lowest errors for wave height and period are for the 0.22-Hz cutoff and the theoretical tail.

Table F-2
Error Statistics for Buoy and Pressure Gauge Comparisons

	Deployment 7		Deployment 8	
Cutoff	rms error in H_{mo} m	rms error in T_p sec	rms error in H_{mo} m	rms error in T_p sec
0.3-Hz cuttoff	0.12	3.4	0.12	2.4
0.22-Hz cutoff	0.14	2.7	0.18	2.1
0.22-Hz cutoff+tail	0.09	2.7	0.11	2.1

Analysis Procedure Selected

Based on comparison of the buoy and pressure gauge spectra, wave heights, and periods, the procedure selected for analysis of the pressure data at Stations 2 and 3 is to apply a spectral cutoff at 0.22 Hz and add a theoretical tail to the spectrum (Equation F2). This procedure minimizes error in the pressure gauge analysis, as evaluated with the buoy measurements. The rms error in wave height using this procedure is 0.09 to 0.11 m, and the rms error in period is 2.1 to 2.7 sec. The wave heights at low tide (when heights are the smallest) tend to be underpredicted, but the results are generally unbiased at other tide levels. The shortest wave period calculated from this procedure is 5 sec. The 0.22-Hz cutoff and theoretical tail were applied for the verification data reported in Chapter 5.

References

Battjes, J. A., and Beji, S. (1992). "Breaking waves propagating over a shoal." *Proceedings 23rd Coastal Engineering Conference*, ASCE, Reston, VA, 42-50.

Miller, H. C., and Vincent, C. L. (1990). "FRF spectrum: TMA with Kitaigordskii's f^{-4} scaling," *Journal of Waterway, Port, Coastal, and Ocean Engineering* 116(1), 57-78.

Wolf, J. (1997). "The analysis of bottom pressure and current data for waves." *Proceedings 7th International Conference on Electronic Engineering in Oceanography*, Southampton, Conference Publication 439, IEE, 165-169.

Appendix G
Bathymetry, Current Patterns, and Bathymetry Change in Design Alternatives[1]

This appendix supplements Chapter 5 of the main text. It provides contour plots of bathymetry, tables of maximum and minimum current speed, vector plots of calculated currents, and contour plots of calculated deposition and erosion within the channels of design alternatives for Willapa Bay. Two sets of vector and bathymetry-change contour plots are provided for each Alternative, one set representing the January 1998 storm and a second set representing fair-weather conditions simulated from 4 September through 6 October 1998.

Contoured bathymetry plots for each Alternative are given in Figures G-1 through G-8. These plots show 1998 bottom topography with alternative channels dug through it. The contour interval is 2 m.

Tables G-1 through G-9 give maximum and minimum current speed calculated within each Region (defined in Figure 6-39) for peak ebb and flood tide. Peak ebb and flood tide were selected for the entire Willapa Bay entrance and all speeds were from the same time.

Figures G-9 through G-26 show sets of vector plots of current patterns. Each set contains 12 snapshots of the current pattern at 1-hr intervals. Vector plots for the January 1998 storm are given during the time interval of peak wave conditions (starting 17 January 1998 0000 GMT) to provide fields of wave-generated currents combined with tide- and wind-generated currents for large waves. Vector plots for fair weather were selected to show current fields for low wave conditions over a tidal cycle so that the ebb and flood currents are depicted in the images. The date of the fair weather plots is 8 September 1998 and time stamps are Greenwich mean time (GMT).

The primary factors controlling currents in an inlet are bottom topography, phase of the tide, and presence and magnitude of waves. These factors interact to form time-varying current patterns and modify the bottom by initiating

[1] Written by Dr. Adele Militello, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS, and by Mr. Mitchell E. Brown, Mevatec Corporation, Vicksburg, MS.

sediment into the water column and depositing it in another location. Waves modify the current by adding momentum and by changing the slope of the water surface. Changes in current patterns by waves and tidal phase are shown in the plots of currents fields for each Alternative (Figures G-9 through G-26). As an example, Figures G-9 and G-18 show sequences of currents during storm and fair weather conditions, respectively, for Alternative 1. During the storm, waves drive a well-developed rotational current in and near the western portion of the alternative. This current moves water toward the northwest over the channel, then redirects toward the southeast bringing water back across the channel in the curved part of the channel. In addition, ebb flow is enhanced by the waves in the channel east of and adjacent to the S-curve. South of the North Channel, eddies form and disintegrate by wave action on the bar. In contrast, in fair weather, currents vary more smoothly with reduced eddies and curvature (Figure G-18). A gyre over the S-curve is present during fair weather, but its size and strength are reduced from that during the storm. For both storm and fair weather, channelized flow occurs in the North Channel, demonstrating control by bottom topography.

Contour plots of bottom elevation change calculated by the sediment transport model described in Chapter 6 are provided in Figures G-27 through G-44. These plots show cumulative deposition and erosion over the simulation interval for the January 1998 storm (15 days) and the fair weather (32 days) calculations. Plots are divided into two panels. Upper panels show deposition (shoaling) in the channel and lower panels show erosion (scour). All contour plots use the same scale for filled contours, which allows for direct comparison between plots.

Table G-1
Maximum and Minimum Velocity in Regions of Alternative 1 for Fair Weather[1] at Peak Ebb and Flood Tidal Current

Region	Max Ebb Speed, m/sec		Min Ebb Speed, m/sec		Max Flood Speed, m/sec		Min Flood Speed, m/sec	
	Along[2]	Across[3]	Along	Across	Along	Across	Along	Across
1	1.0	--	0.7	--	0.7	--	0.5	--
2	1.3	1.6	0.5	0.3	0.8	0.9	0.3	0.5
3	2.5	1.6	1.5	1.1	1.6	1.0	1.1	0.7
4	2.6	--	1.5	--	1.6	--	1.2	--
5	2.0	--	1.4	--	1.5	--	1.1	--

[1] September 1998 fair weather simulation, duration = 31 days.
[2] Along-channel is defined as current directed at angles ranging from 0 to 45 deg relative to channel axis.
[3] Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis.

Table G-2
Maximum and Minimum Velocity in Regions of Alternative 3A for Fair Weather[1] at Peak Ebb and Flood Tidal Current

Region	Max Ebb Speed, m/sec		Min Ebb Speed, m/sec		Max Flood Speed, m/sec		Min Flood Speed, m/sec	
	Along[2]	Across[3]	Along	Across	Along	Across	Along	Across
1	1.0	--	0.6	--	0.5	--	0.3	--
2	1.1	--	0.8	--	0.7	0.7	0.5	0.7
3	2.6	--	1.1	--	1.3	0.7	0.8	0.7
4	2.4	--	1.8	--	1.3	--	1.1	--
5	2.0	--	1.7	--	1.3	--	1.2	--

[1] September 1998 fair weather simulation, duration = 31 days.

[2] Along-channel is defined as current directed at angles ranging from 0 to 45 deg relative to channel axis.

[3] Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis.

Table G-3
Maximum and Minimum Velocity in Regions of Alternative 3B for Fair Weather[1] at Peak Ebb and Flood Tidal Current

Region	Max Ebb Speed, m/sec		Min Ebb Speed, m/sec		Max Flood Speed, m/sec		Min Flood Speed, m/sec	
	Along[2]	Across[3]	Along	Across	Along	Across	Along	Across
1	1.0	--	0.7	--	0.7	--	0.5	--
2	1.1	1.6	0.4	0.4	0.7	1.0	0.2	0.5
3	2.6	1.3	1.6	1.1	1.4	0.9	1.0	0.6
4	2.6	--	1.8	--	1.4	--	1.1	--
5	2.0	--	1.7	--	1.3	--	1.2	--

[1] September 1998 fair weather simulation, duration = 31 days.

[2] Along-channel is defined as current directed at angles ranging from 0 to 45 deg relative to channel axis.

[3] Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis.

Table G-4
Maximum and Minimum Velocity in Regions of Alternative 3F for Fair Weather[1] at Peak Ebb and Flood Tidal Current

Region	Max Ebb Speed, m/sec		Min Ebb Speed, m/sec		Max Flood Speed, m/sec		Min Flood Speed, m/sec	
	Along[2]	Across[3]	Along	Across	Along	Across	Along	Across
1	1.1	--	0.7	--	0.5	--	0.3	--
2	1.2	--	0.9	--	0.7	--	0.4	--
3	2.6	--	1.2	--	1.4	--	0.7	--
4	2.5	--	1.8	--	1.3	--	1.1	--
5	2.0	--	1.6	--	1.3	--	1.2	--

[1] September 1998 fair weather simulation, duration = 31 days.

[2] Along-channel is defined as current directed at angles ranging from 0 to 45 deg relative to channel axis.

[3] Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis.

Table G-5
Maximum and Minimum Velocity in Regions of Alternative 3G for Fair Weather[1] at Peak Ebb and Flood Tidal Current

Region	Max Ebb Speed, m/sec		Min Ebb Speed, m/sec		Max Flood Speed, m/sec		Min Flood Speed, m/sec	
	Along[2]	Across[3]	Along	Across	Along	Across	Along	Across
1	1.1	--	0.3	--	0.7	0.3	0.3	0.2
2	1.2	1.5	0.4	0.4	0.7	1.0	0.2	0.2
3	2.3	--	1.1	--	1.3	0.9	0.6	0.5
4	2.4	--	1.6	--	0.9	--	1.3	--
5	2.1	--	1.5	--	1.3	--	1.0	--

[1] September 1998 fair weather simulation, duration = 31 days.

[2] Along-channel is defined as current directed at angles ranging from 0 to 45 deg relative to channel axis.

[3] Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis.

Table G-6
Maximum and Minimum Velocity in Regions of Alternative 3H-a for Fair Weather[1] at Peak Ebb and Flood Tidal Current

Region	Max Ebb Speed, m/sec		Min Ebb Speed, m/sec		Max Flood Speed, m/sec		Min Flood Speed, m/sec	
	Along[2]	Across[3]	Along	Across	Along	Across	Along	Across
1	1.0	0.8	0.9	0.7	0.7	0.6	0.5	0.5
2	1.2	0.8	0.5	0.7	0.9	0.4	0.5	0.4
3	2.1	--	1.2	--	1.3	0.8	0.8	0.8
4	2.0	--	1.5	--	1.3	--	1.1	--
5	2.0	--	1.6	--	1.5	--	1.3	--

[1] September 1998 fair weather simulation, duration = 31 days.

[2] Along-channel is defined as current directed at angles ranging from 0 to 45 deg relative to channel axis.

[3] Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis.

Table G-7
Maximum and Minimum Velocity in Regions of Alternative 3H-b for Fair Weather[1] at Peak Ebb and Flood Tidal Current

Region	Max Ebb Speed, m/sec		Min Ebb Speed, m/sec		Max Flood Speed, m/sec		Min Flood Speed, m/sec	
	Along[2]	Across[3]	Along	Across	Along	Across	Along	Across
1	1.0	0.9	0.9	0.7	0.7	0.7	0.5	0.6
2	1.1	0.9	0.6	0.6	1.2	0.4	0.5	0.4
3	1.9	--	1.1	--	1.3	0.7	0.7	0.7
4	1.9	--	1.5	--	1.2	--	0.8	--
5	1.7	2.3	1.6	1.7	1.3	1.7	1.1	1.3

[1] September 1998 fair weather simulation, duration = 31 days.

[2] Along-channel is defined as current directed at angles ranging from 0 to 45 deg relative to channel axis.

[3] Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis.

Table G-8
Maximum and Minimum Velocity in Regions of Alternative 4A for Fair Weather[1] at Peak Ebb and Flood Tidal Current

Region	Max Ebb Speed, m/sec		Min Ebb Speed, m/sec		Max Flood Speed, m/sec		Min Flood Speed, m/sec	
	Along[2]	Across[3]	Along	Across	Along	Across	Along	Across
1	0.4	--	0.3	--	0.5	--	0.3	--
2	2.7	--	0.4	--	1.1	--	0.5	--
3	2.8	--	1.1	--	1.1	--	0.8	--
4	2.0	--	1.2	--	1.8	--	1.0	--
5	NA[4]	NA	NA	NA	NA	NA	NA	NA

[1] September 1998 fair weather simulation, duration = 31 days.

[2] Along-channel is defined as current directed at angles ranging from 0 to 45 deg relative to channel axis.

[3] Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis.

[4] Alternative 4A does not extend into Region 5.

Table G-9
Maximum and Minimum Velocity in Regions of Alternative 4E for Fair Weather[1] at Peak Ebb and Flood Tidal Current

Region	Max Ebb Speed, m/sec		Min Ebb Speed, m/sec		Max Flood Speed, m/sec		Min Flood Speed, m/sec	
	Along[2]	Across[3]	Along	Across	Along	Across	Along	Across
1	0.4	--	0.3	--	0.4	--	0.4	--
2	1.6	1.8	0.3	0.4	1.0	0.6	0.4	0.6
3	1.3	1.6	0.8	0.8	1.2	--	0.8	--
4	1.9	--	1.0	--	1.8	--	0.9	--
5	1.3	1.5	0.9	1.2	1.4	1.4	1.0	1.2

[1] September 1998 fair weather simulation, duration = 31 days.

[2] Along-channel is defined as current directed at angles ranging from 0 to 45 deg relative to channel axis.

[3] Across-channel is defined as current directed at angles ranging from 45 to 90 deg relative to channel axis.

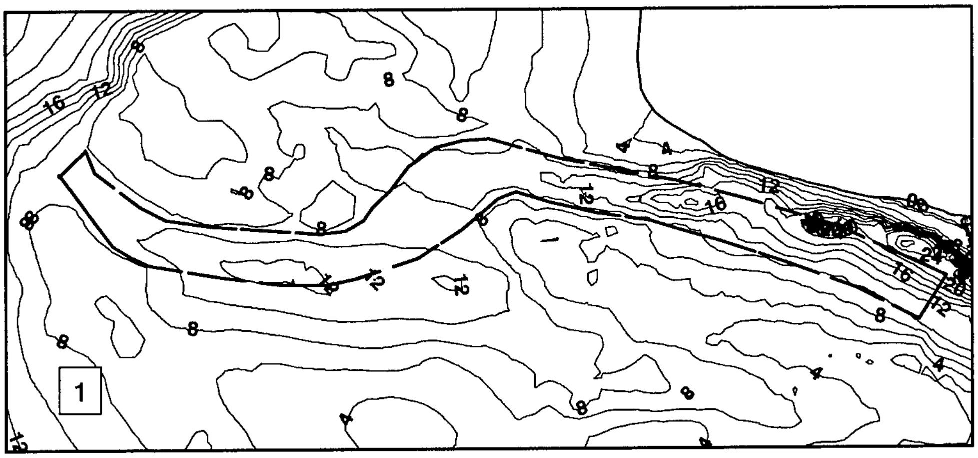


Figure G-1. Bathymetry for Alternative 1

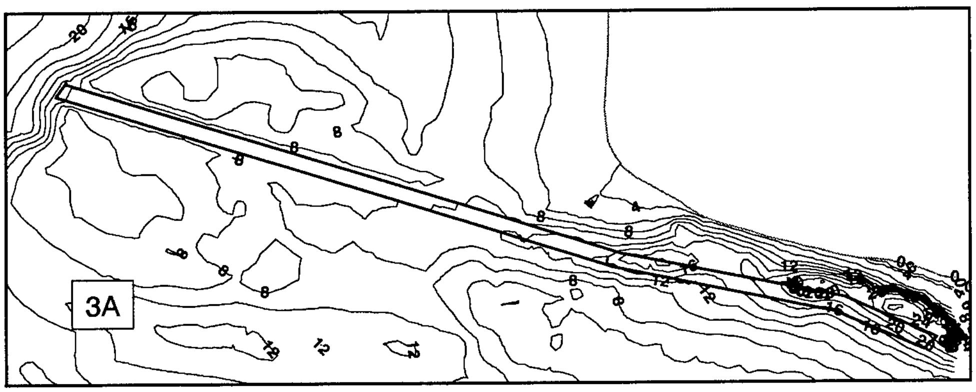


Figure G-2. Bathymetry for Alternative 3A

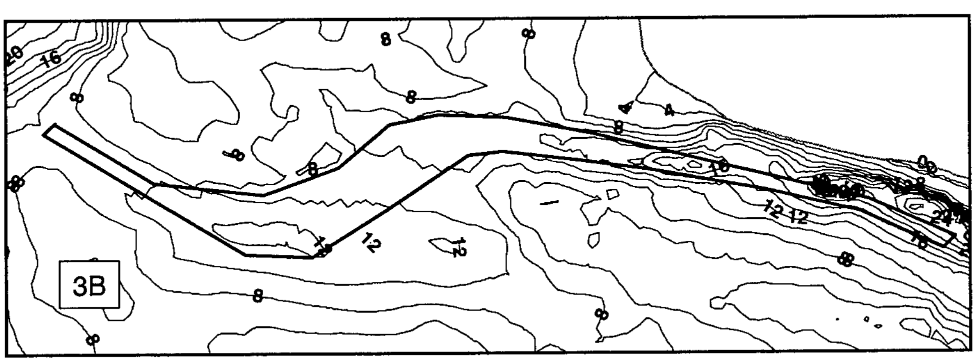


Figure G-3. Bathymetry for Alternative 3B

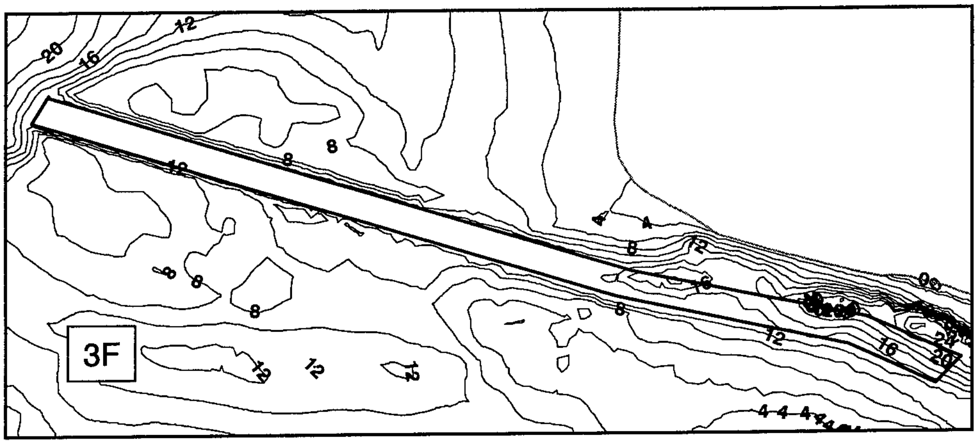


Figure G-4. Bathymetry for Alternative 3F

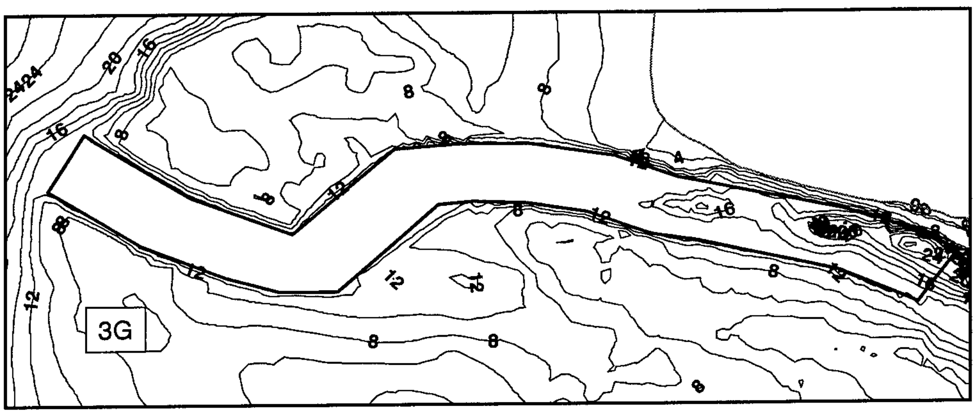


Figure G-5. Bathymetry for Alternative 3G

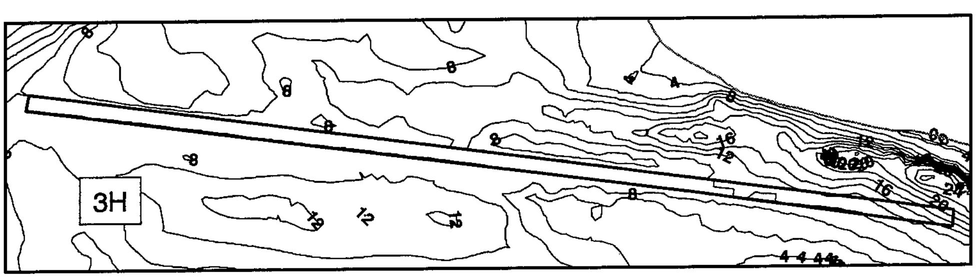


Figure G-6. Bathymetry for Alternative 3H (3H-a and 3H-b)

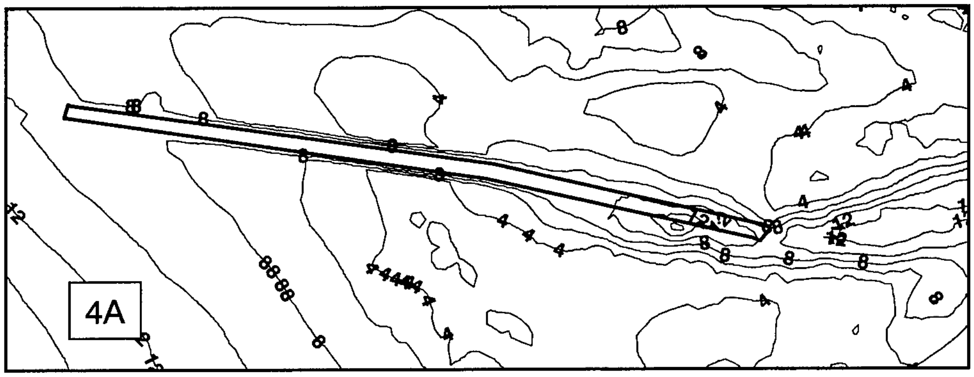


Figure G-7. Bathymetry for Alternative 4A

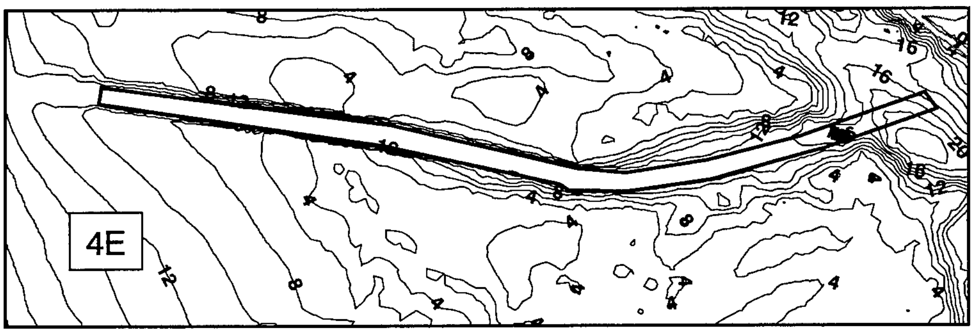


Figure G-8. Bathymetry for Alternative 4E

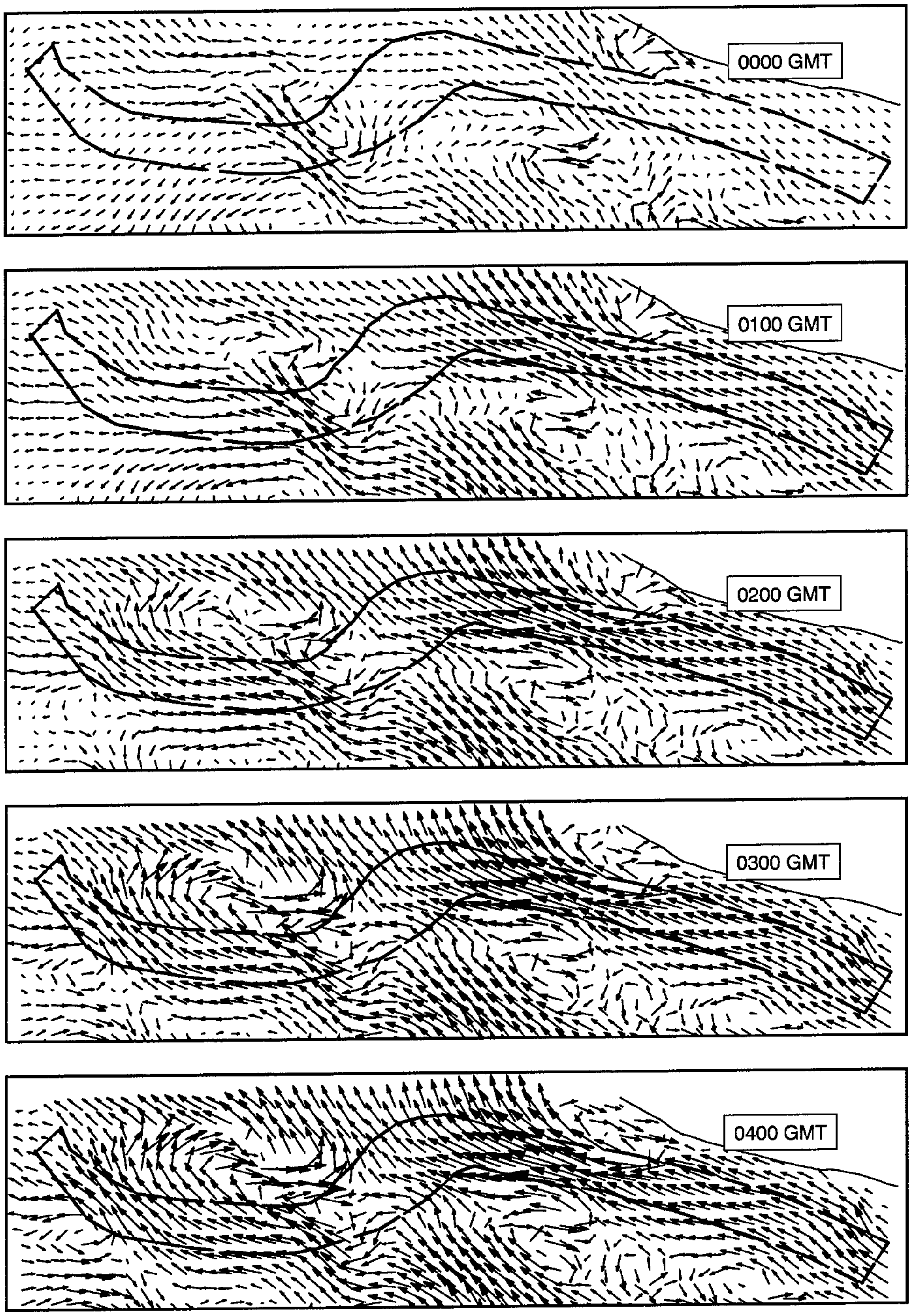


Figure G-9. Current patterns in Alternative 1 channel, January 1998 storm (Sheet 1 of 3)

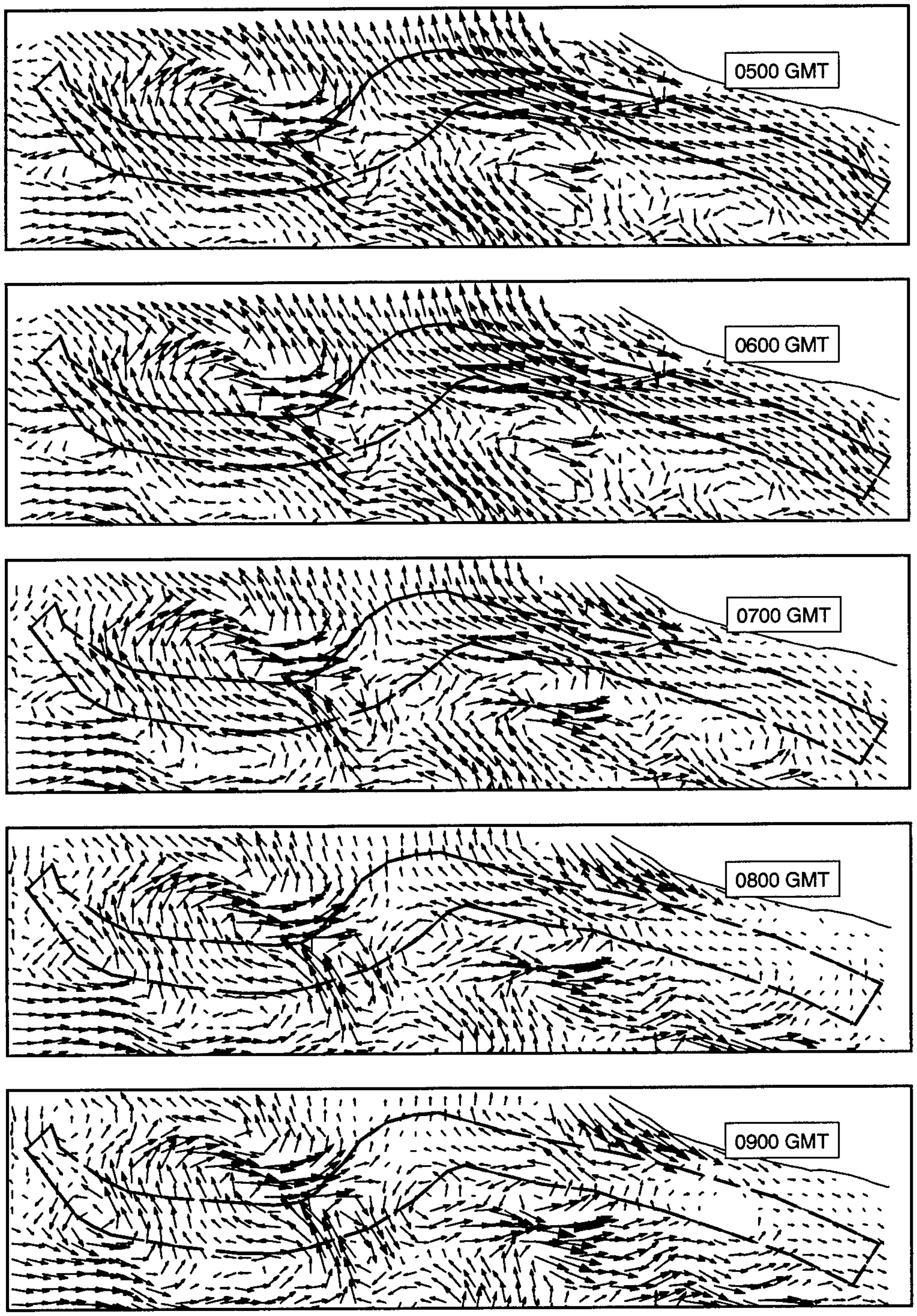
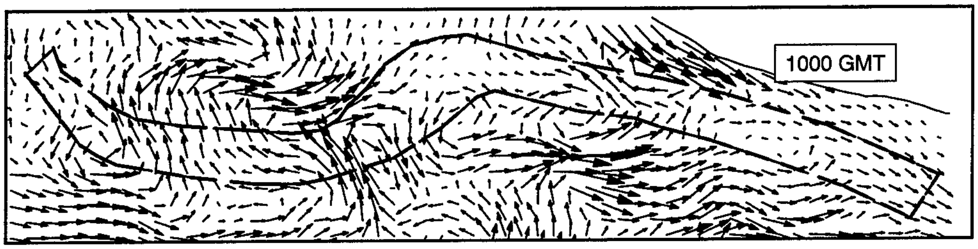


Figure G-9. (Sheet 2 of 3)



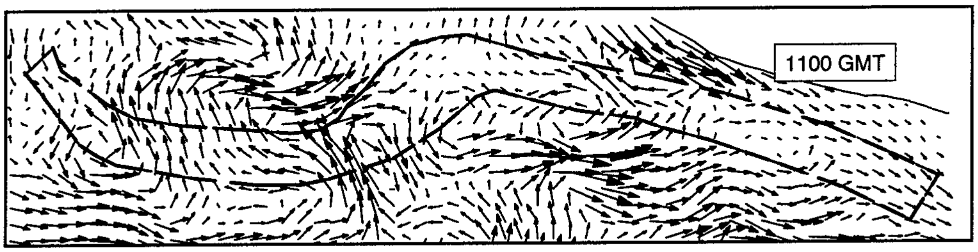


Figure G-9. (Sheet 3 of 3)

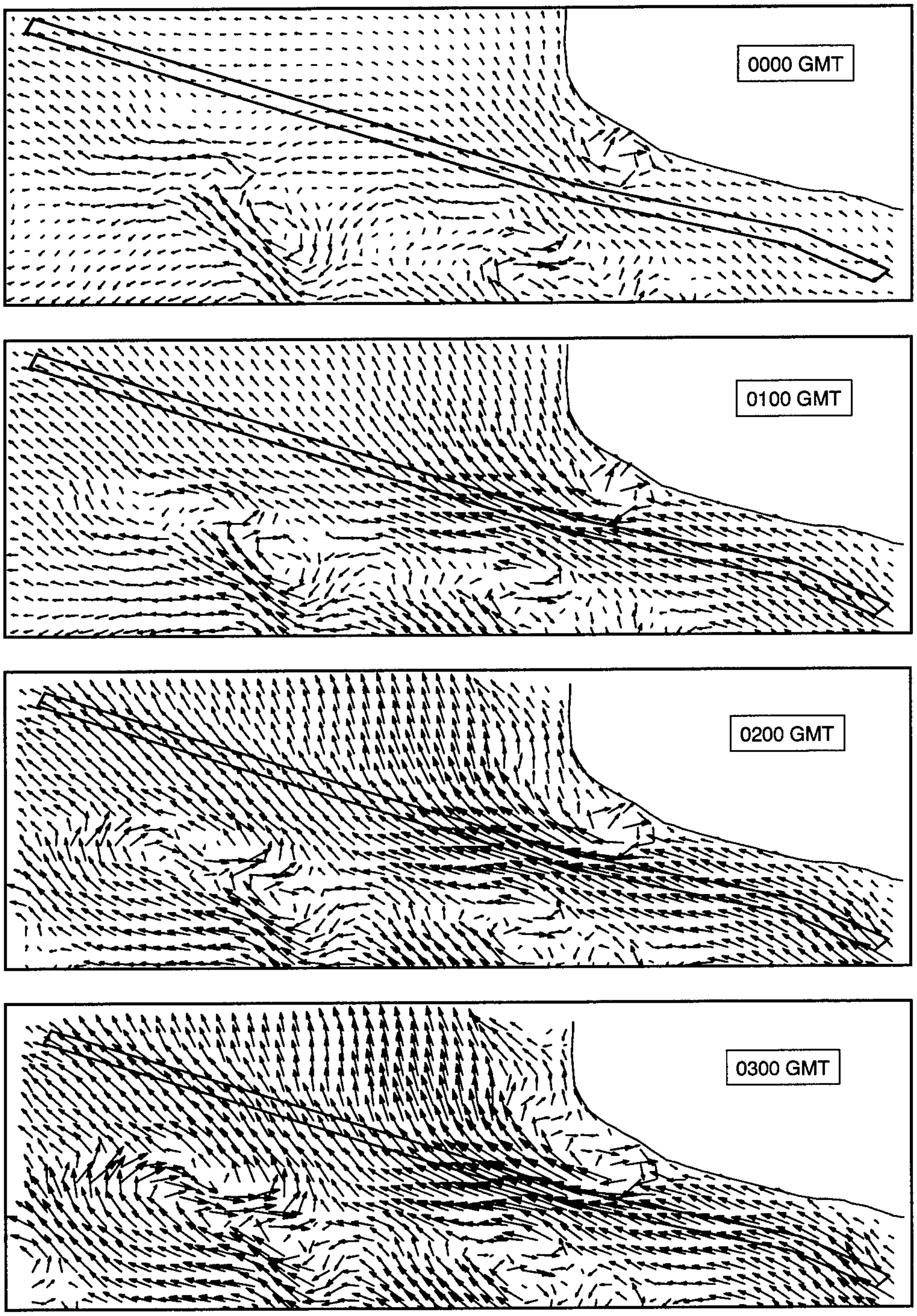


Figure G-10. Current patterns in Alternative 3A channel, January 1998 storm (Sheet 1 of 3)

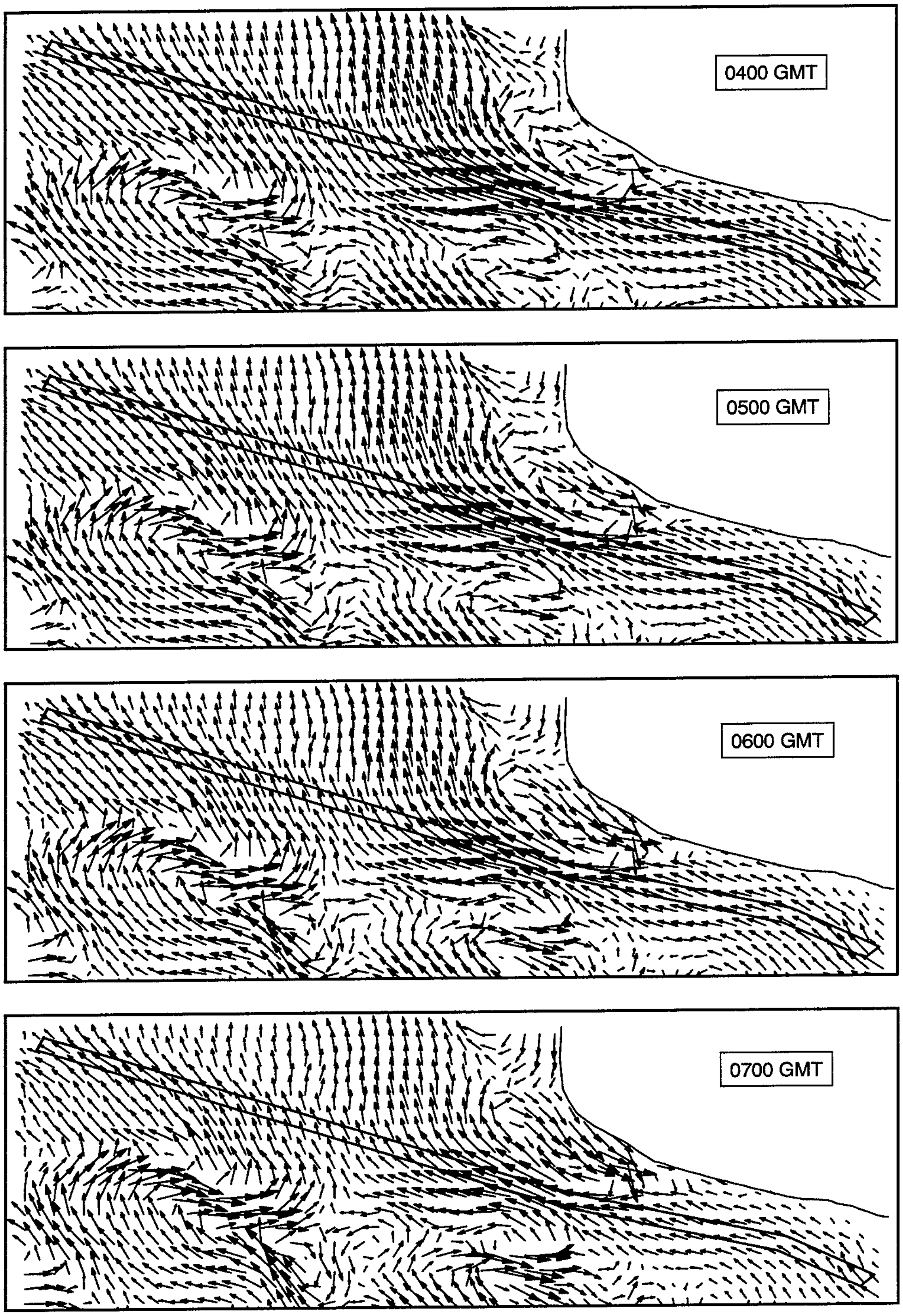


Figure G-10. (Sheet 2 of 3)

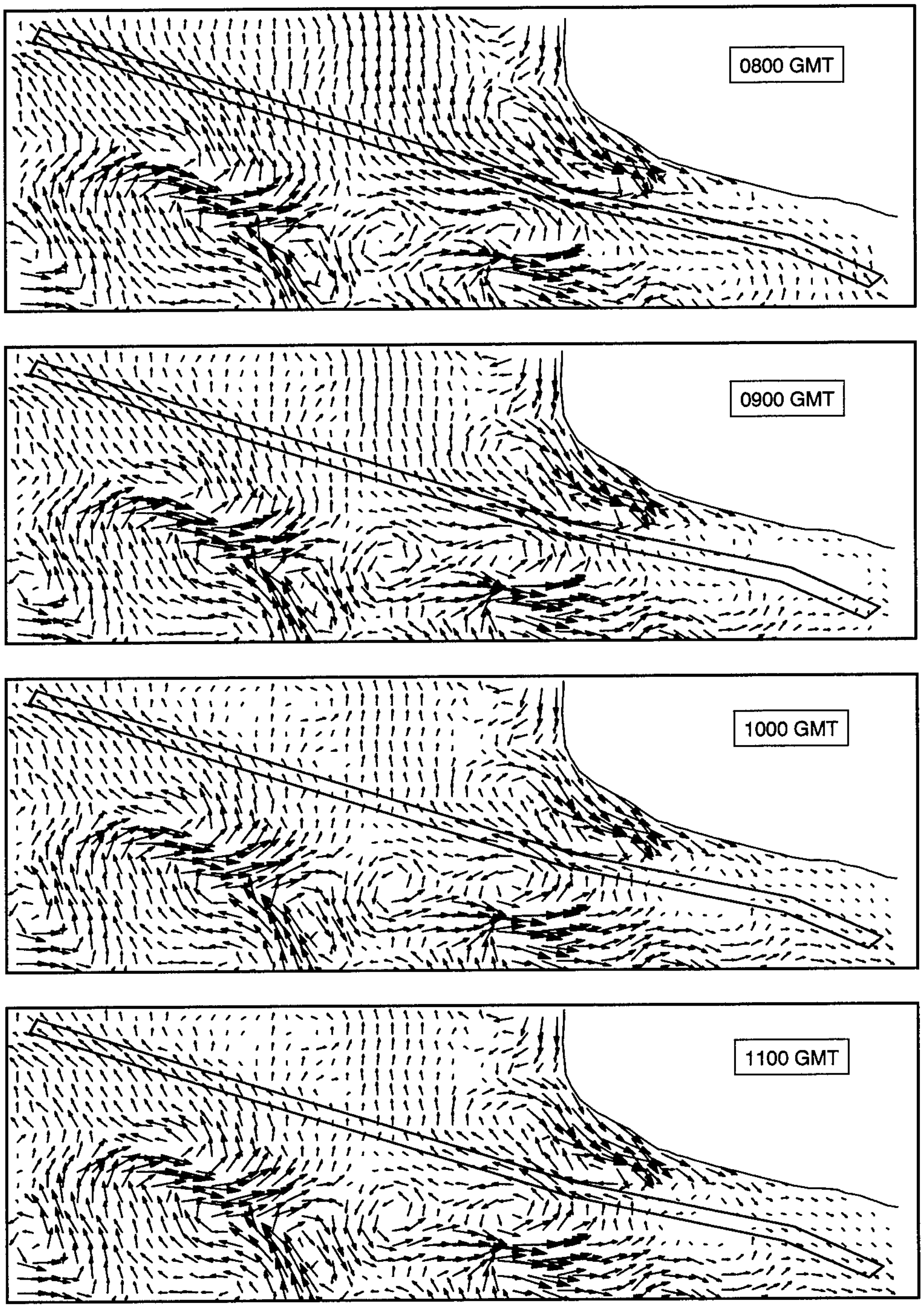


Figure G-10. (Sheet 3 of 3)

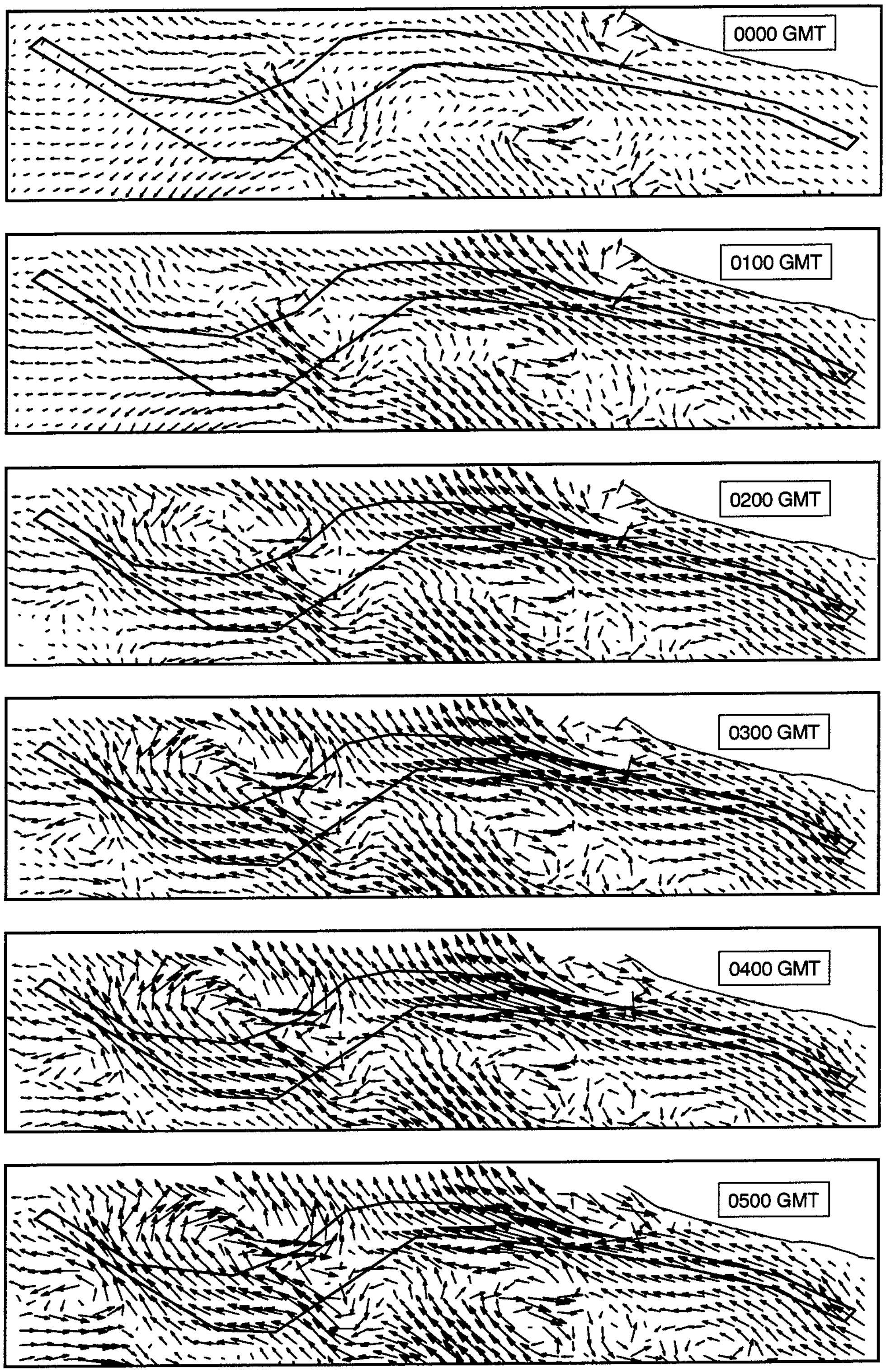


Figure G-11. Current patterns in Alternative 3B channel, January 1998 storm (Continued)

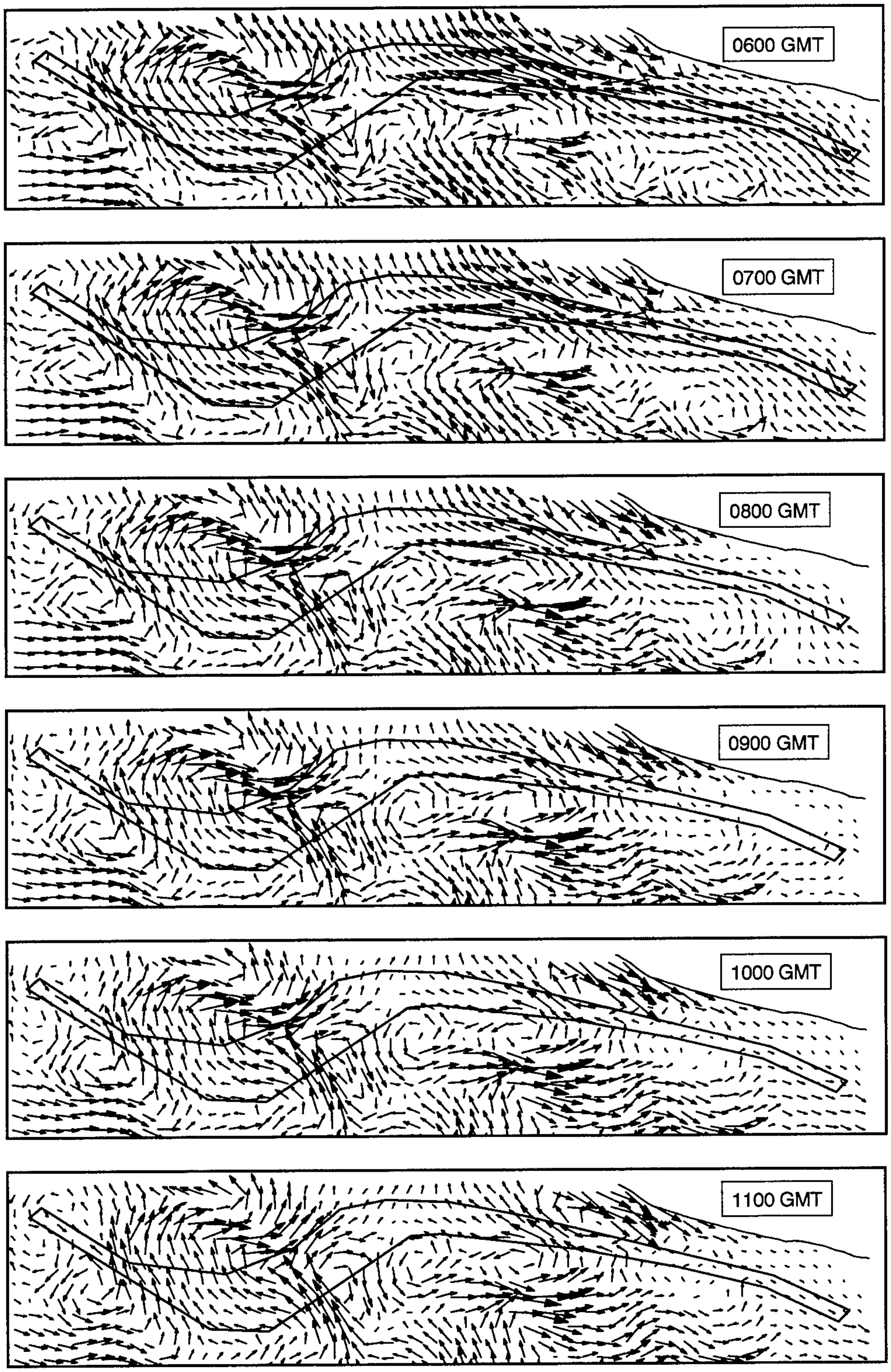


Figure G-11. (Concluded)

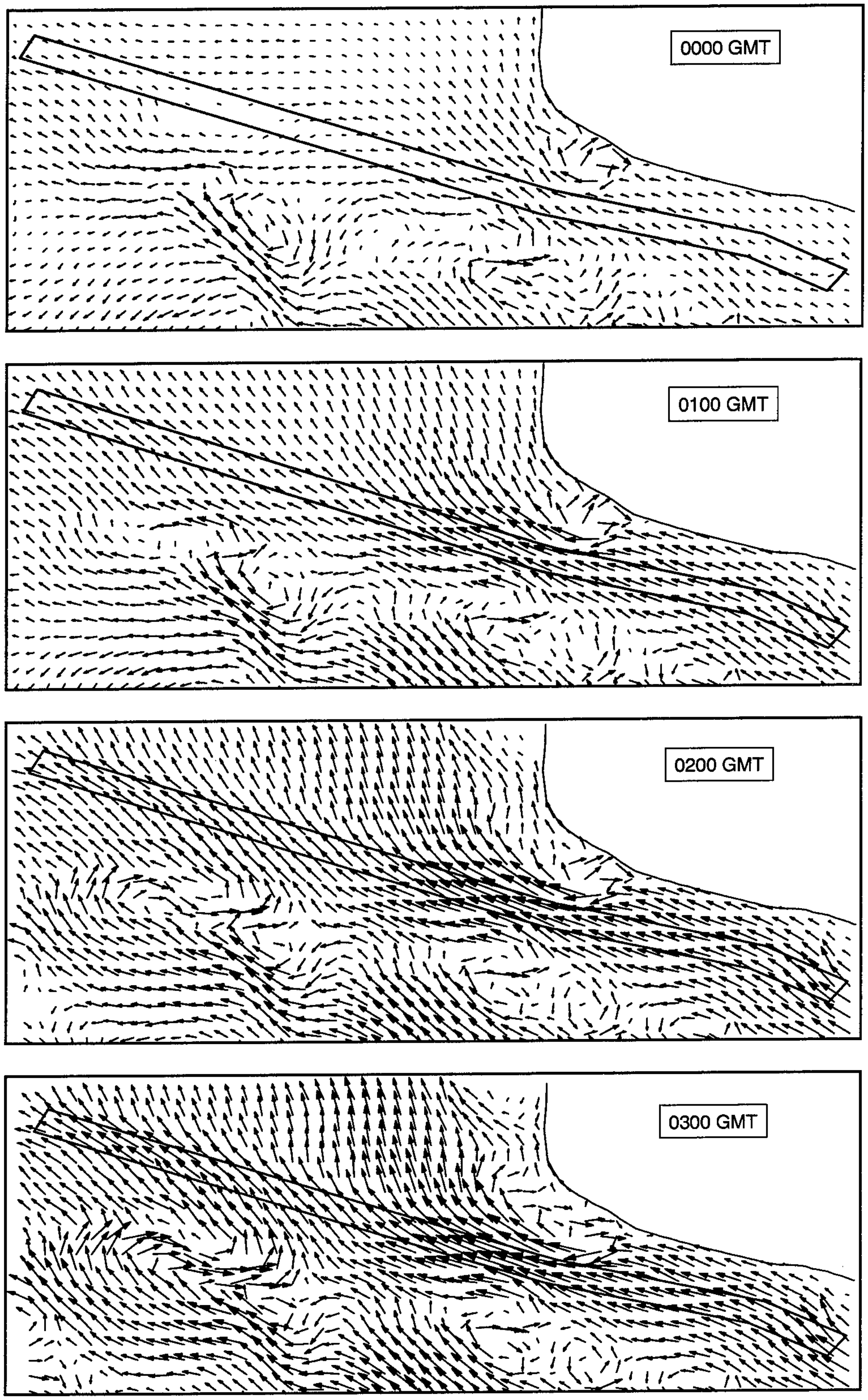


Figure G-12. Current patterns in Alternative 3F channel, January 1998 storm (Sheet 1 of 3)

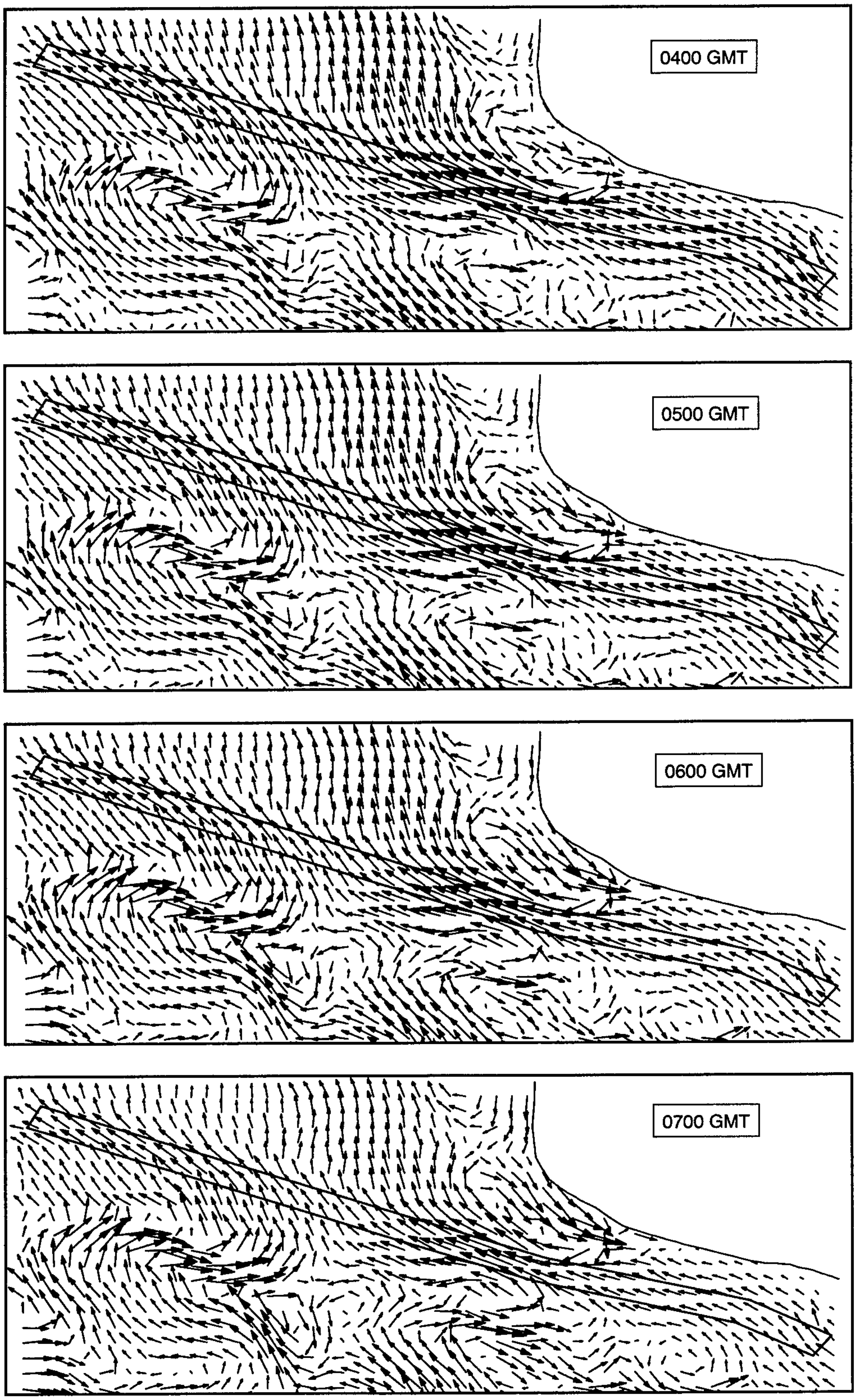


Figure G-12. (Sheet 2 of 3)

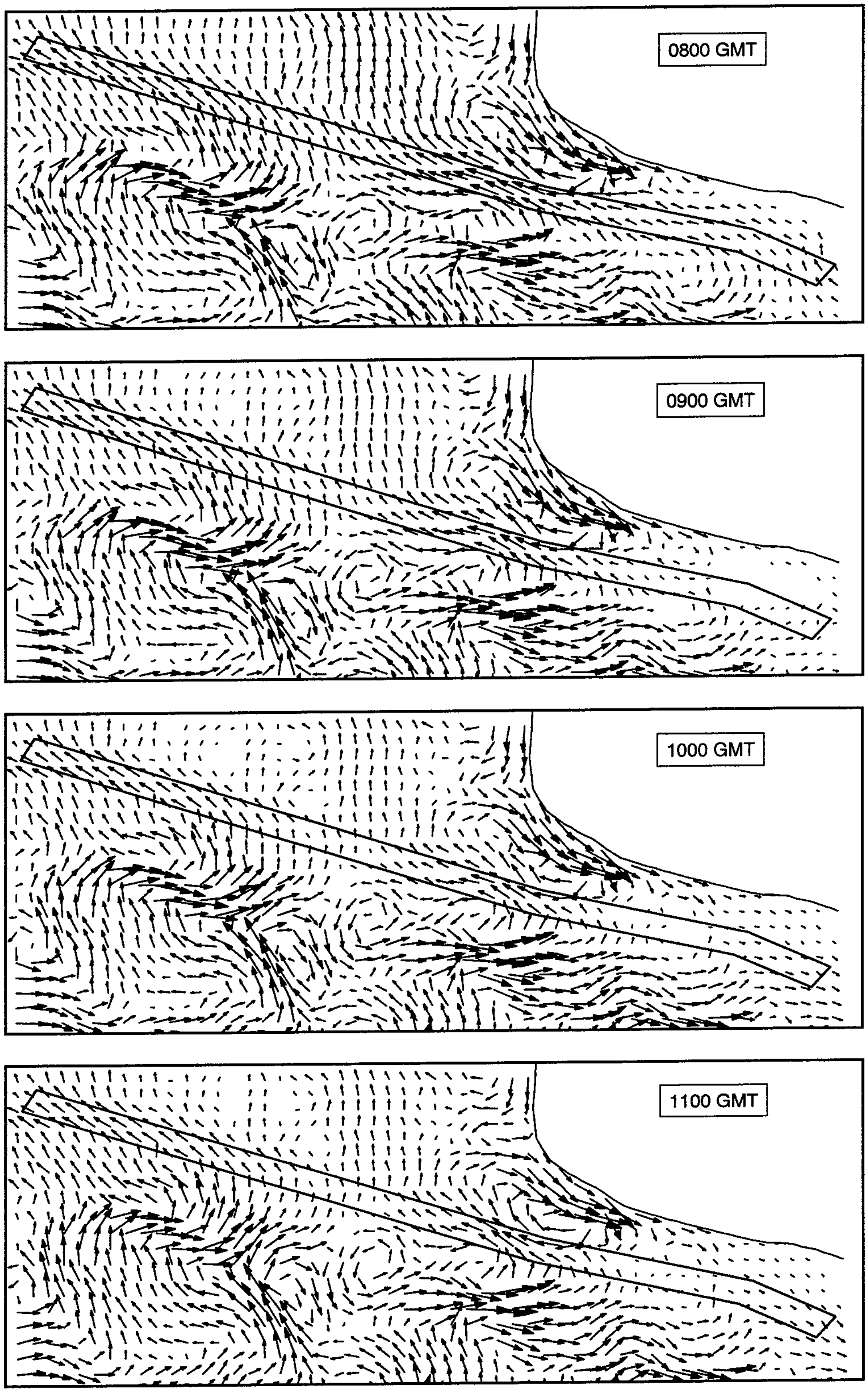


Figure G-12. (Sheet 3 of 3)

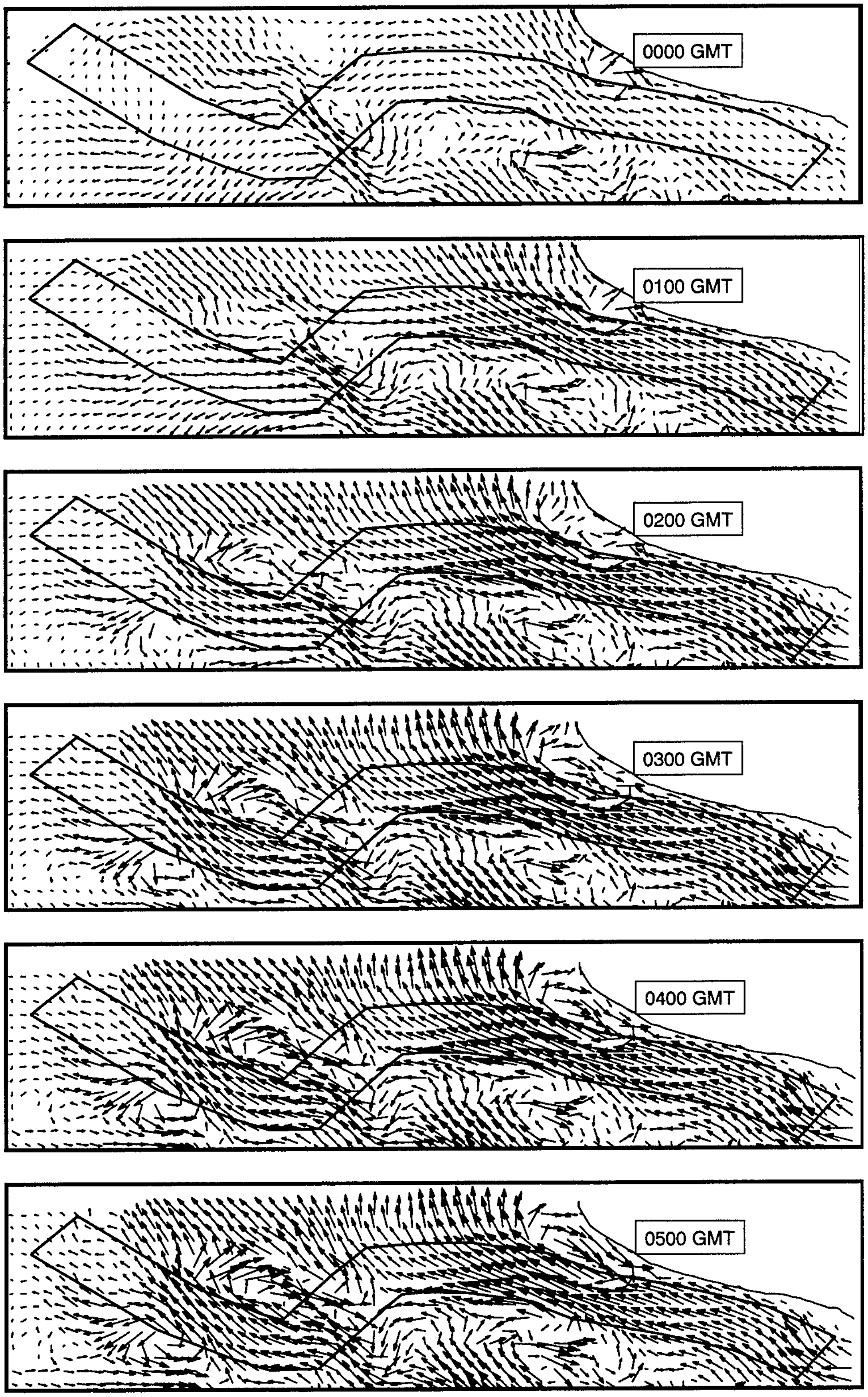


Figure G-13. Current patterns in Alternative 3G channel, January 1998 storm (Continued)

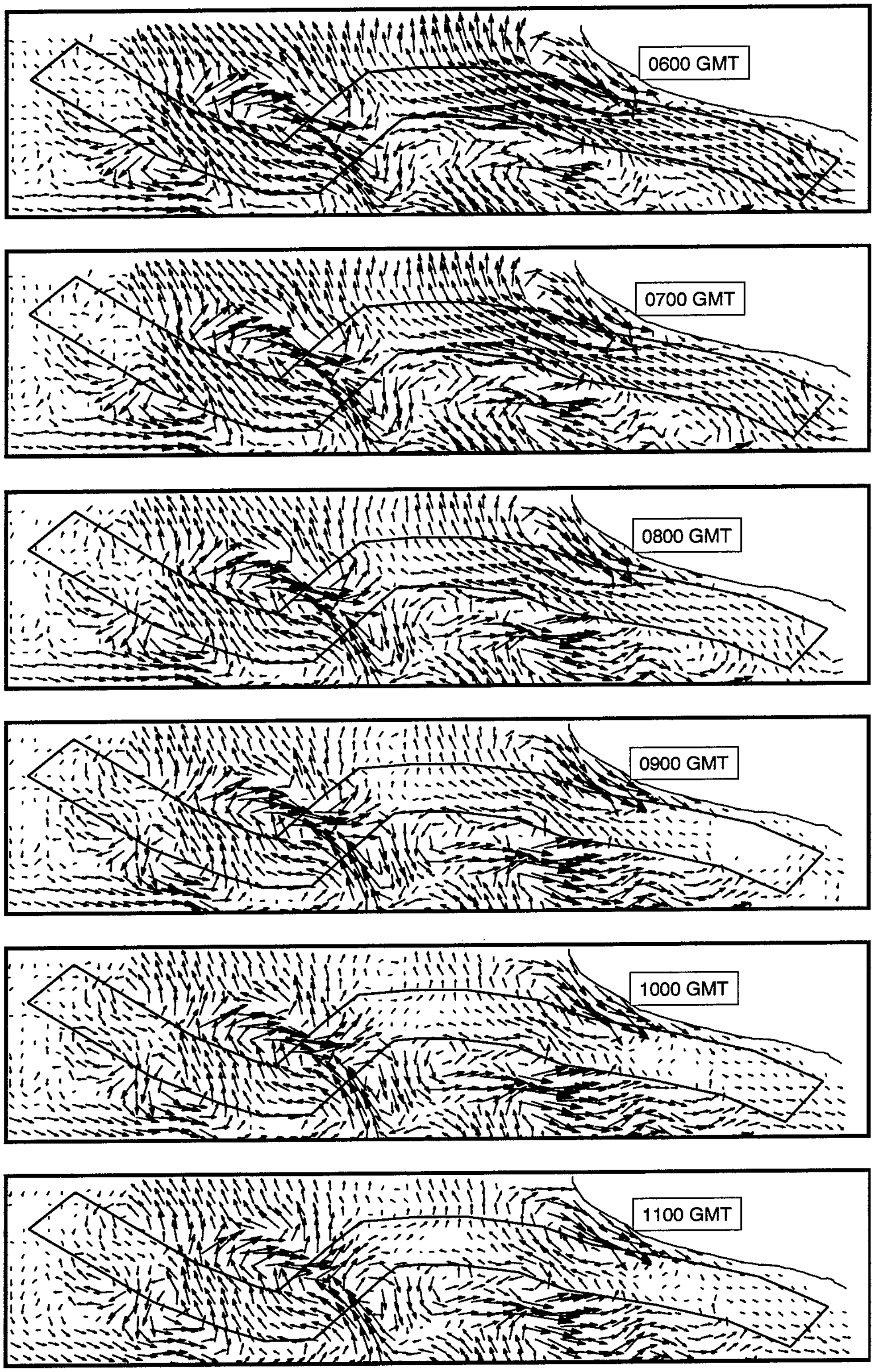


Figure G-13. (Concluded)

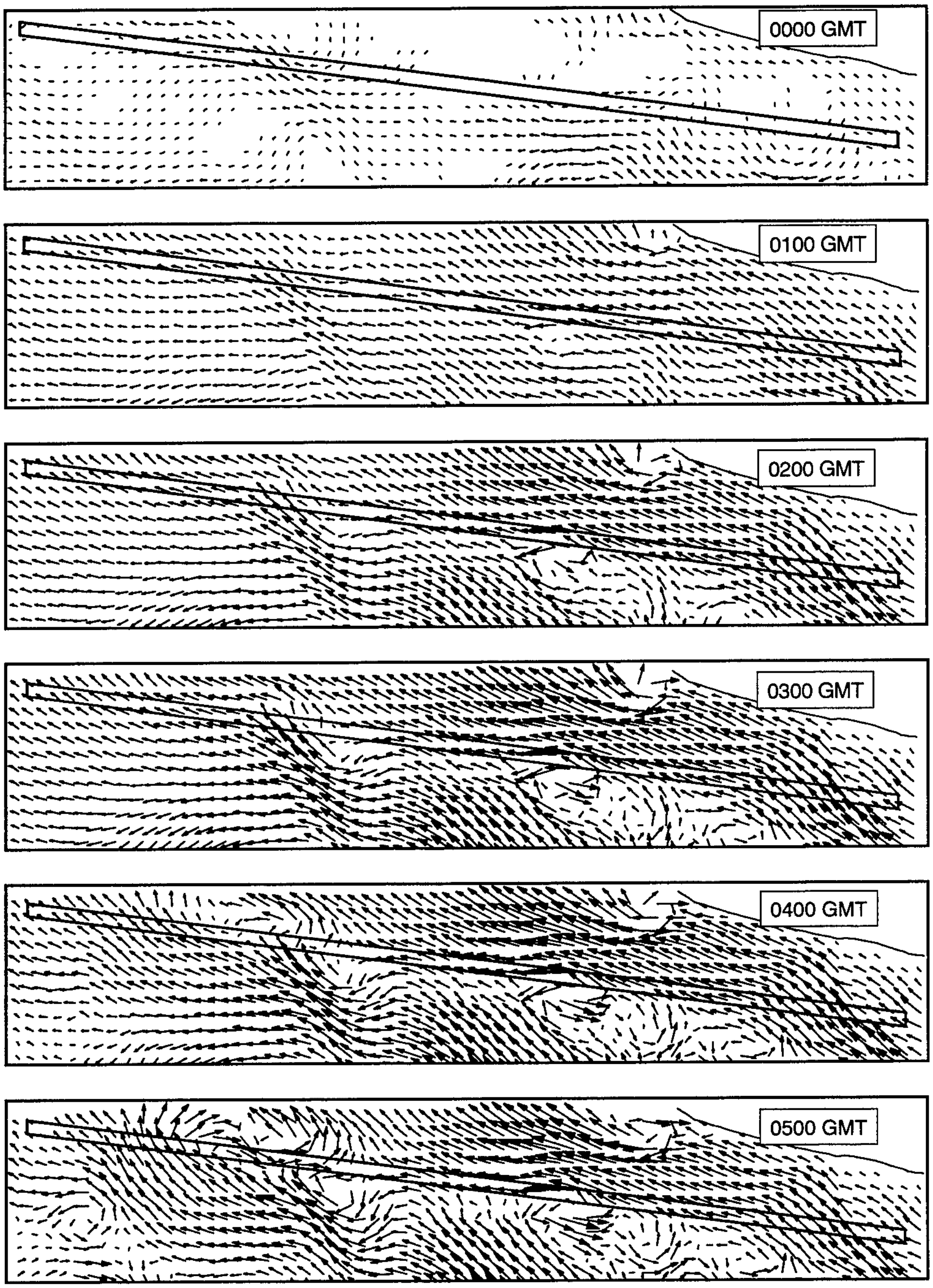
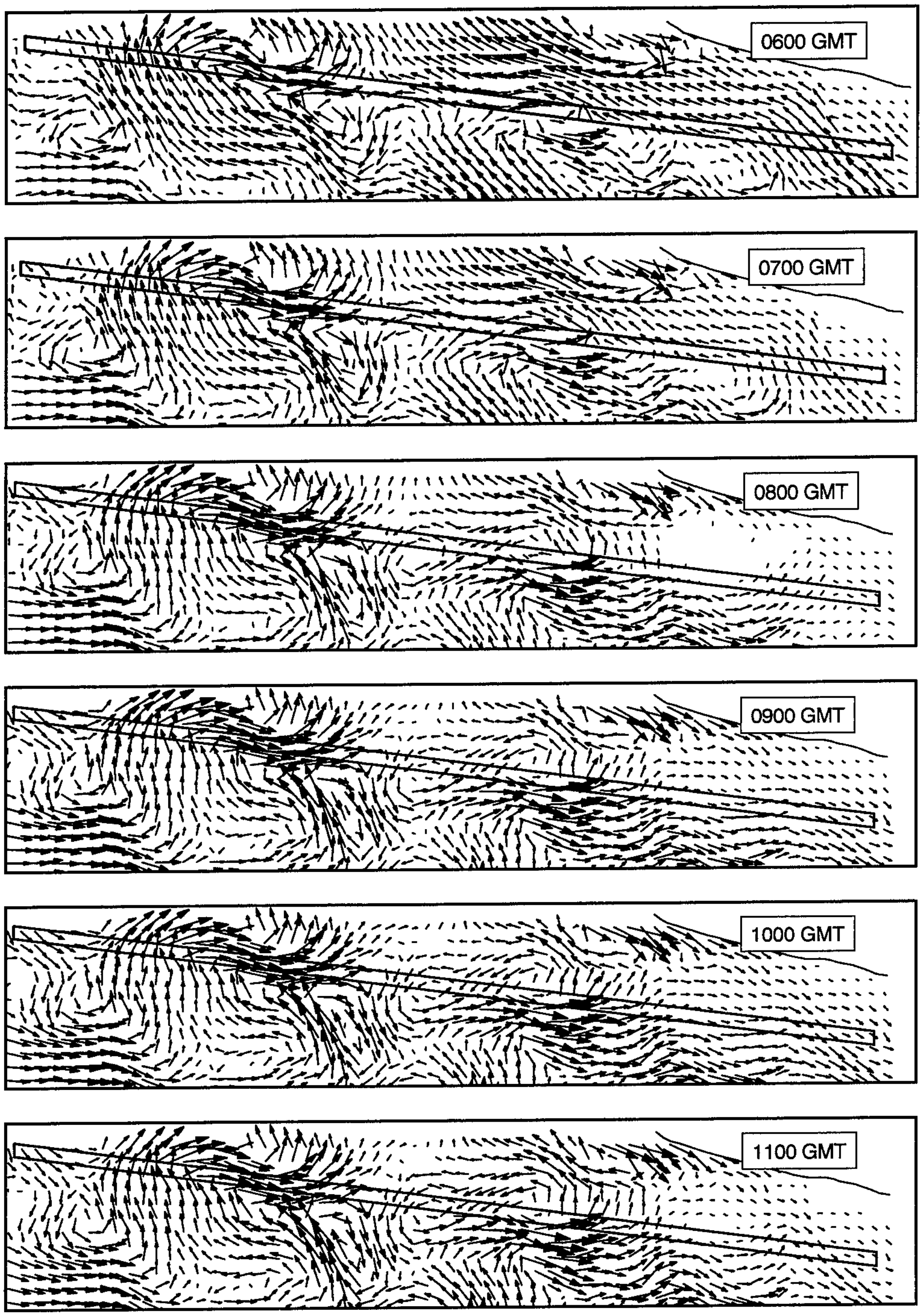


Figure G-14. Current patterns in Alternative 3H-a channel, January 1998 storm (Continued)



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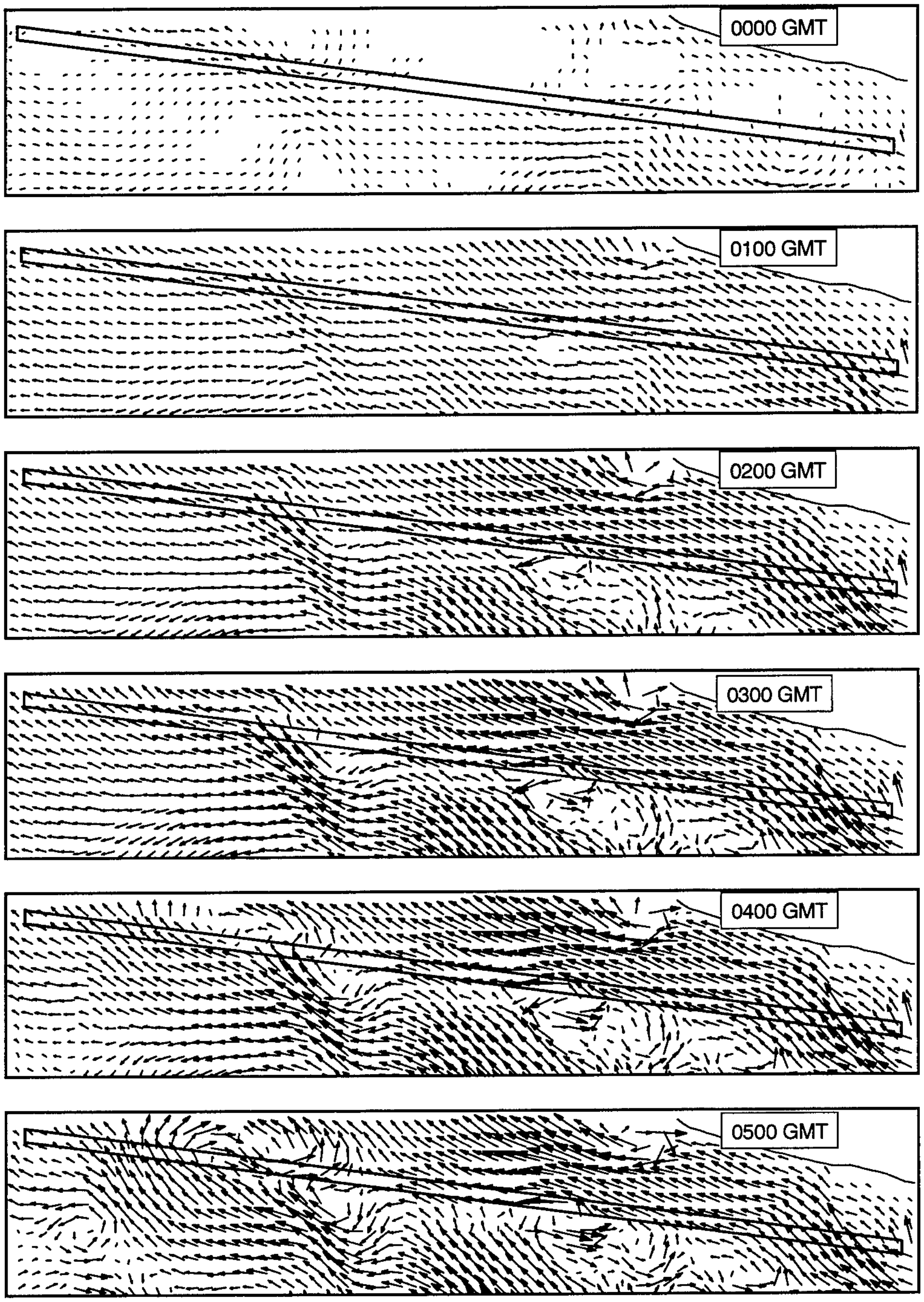


Figure G-15. Current patterns in Alternative 3H-b channel, January 1998 storm (Continued)

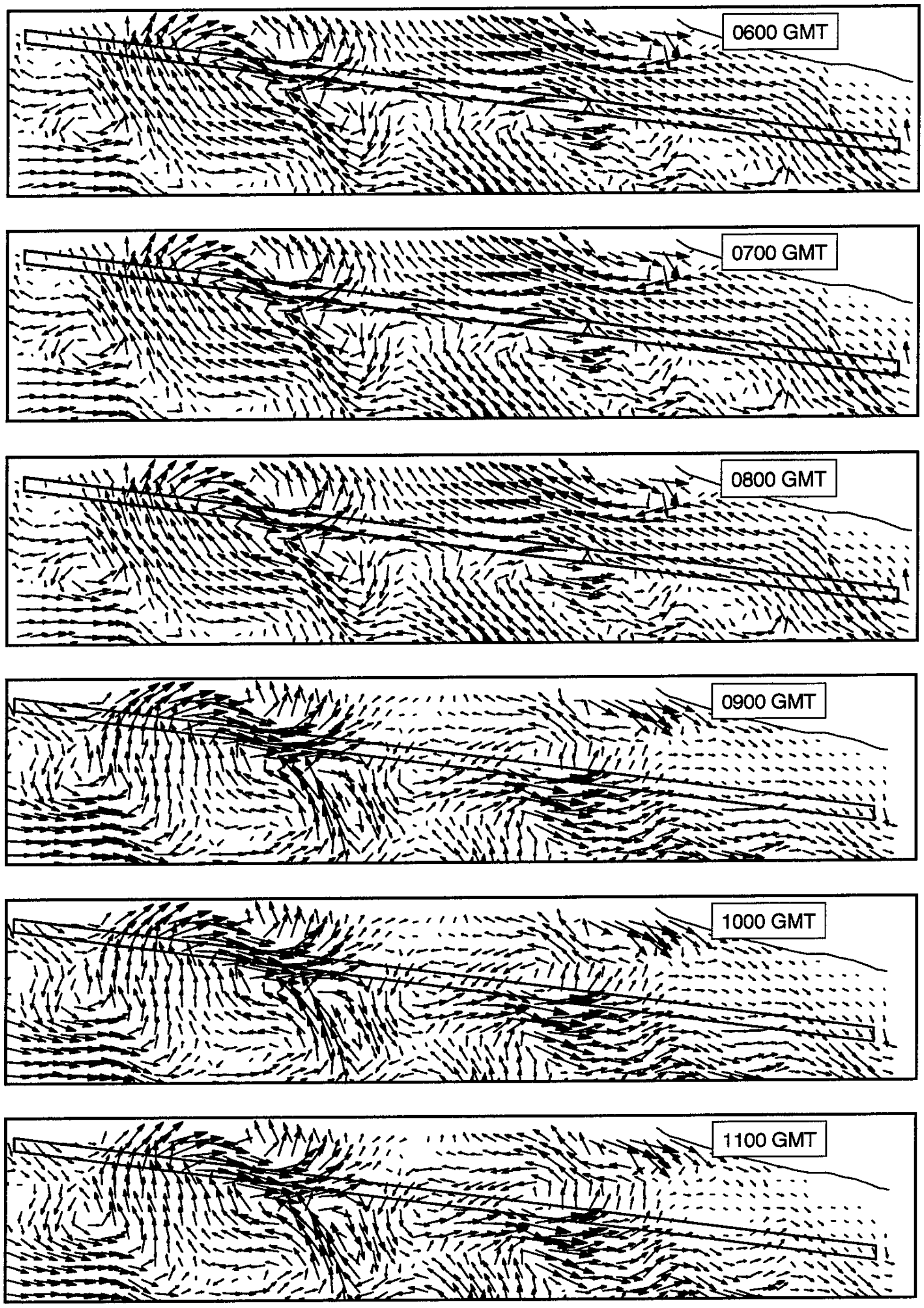


Figure G-15. (Concluded)

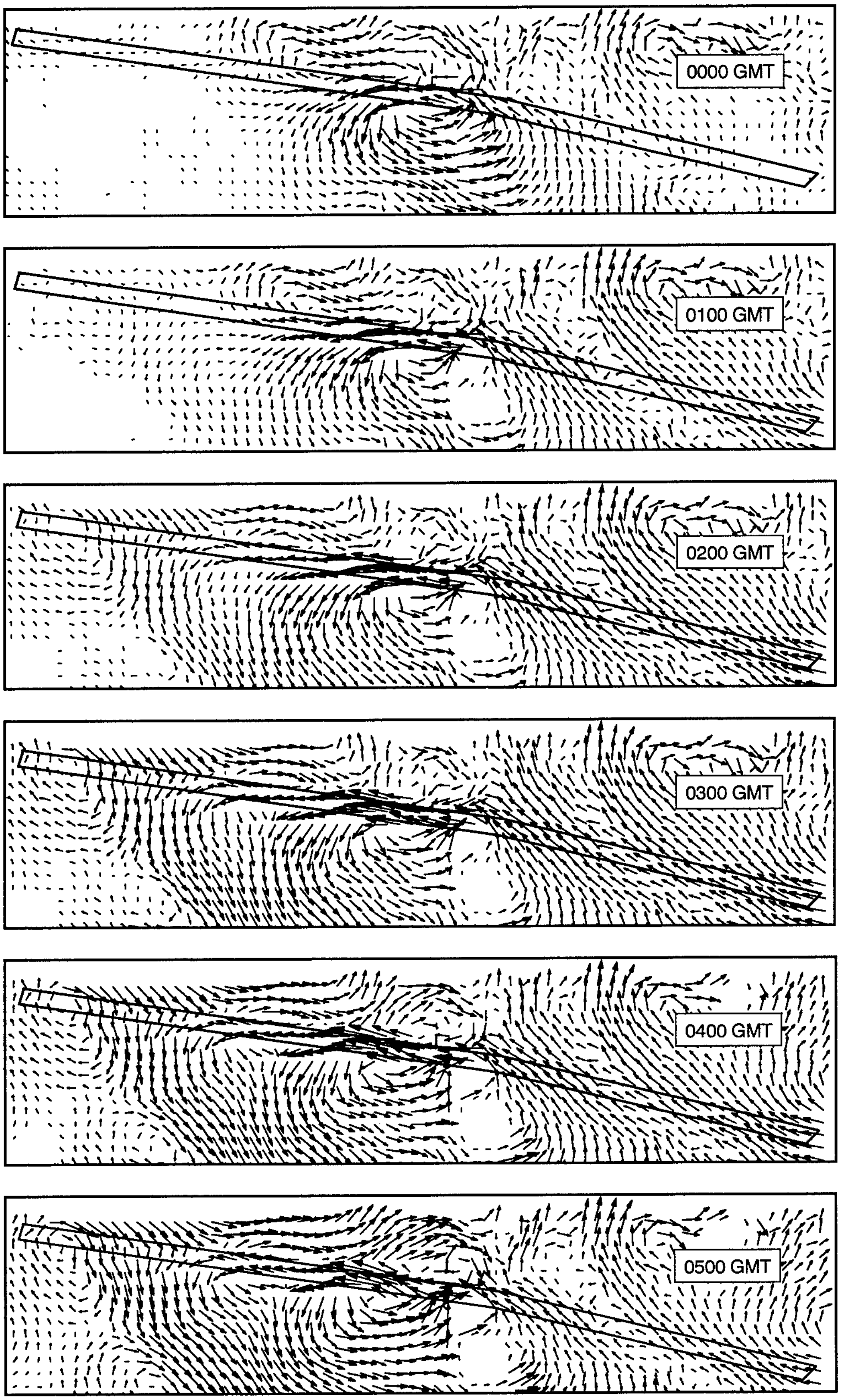


Figure G-16. Current patterns in Alternative 4A channel, January 1998 storm (Continued)

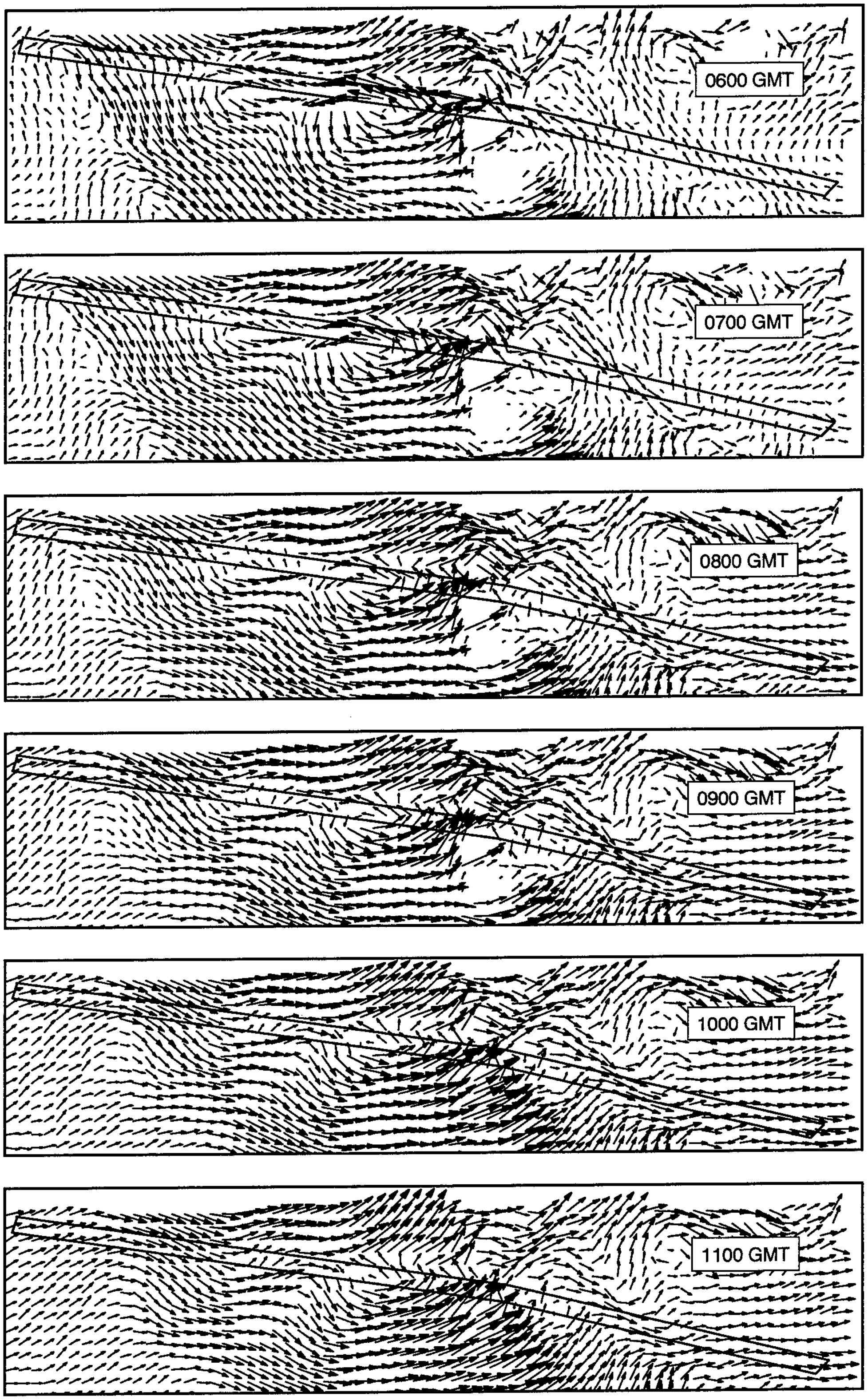


Figure G-16. (Concluded)

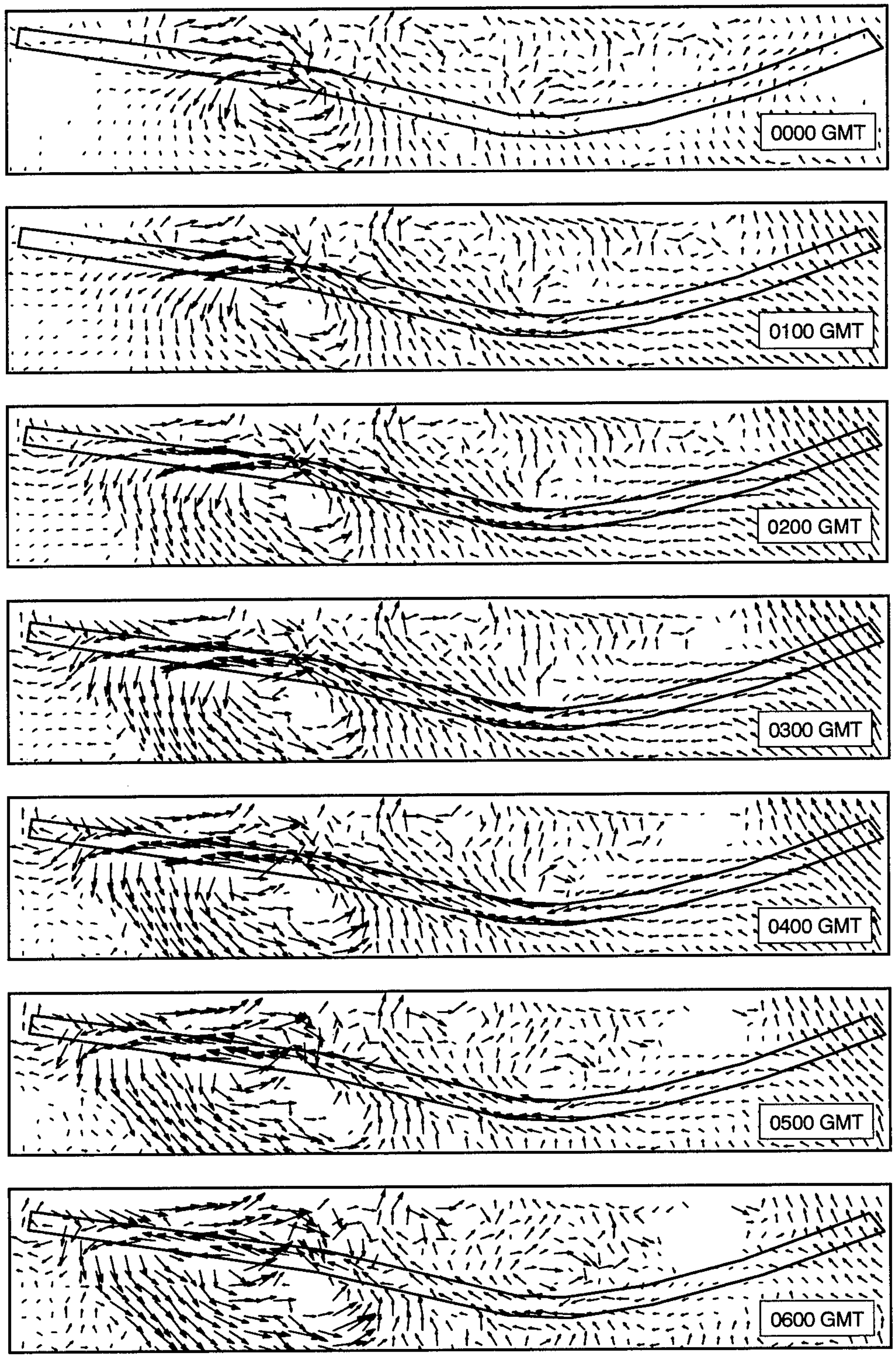


Figure G-17. Current patterns in Alternative 4E channel, January 1998 storm (Continued)

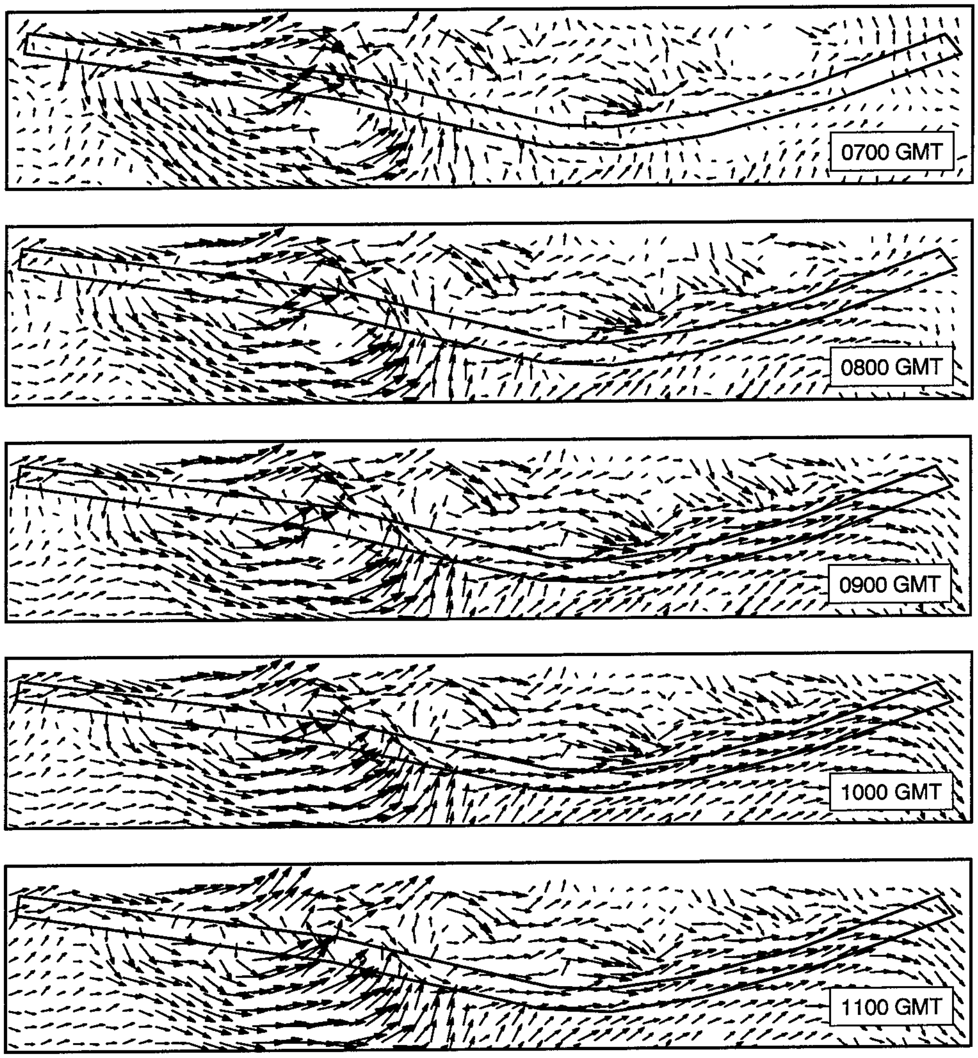


Figure G-17. (Concluded)

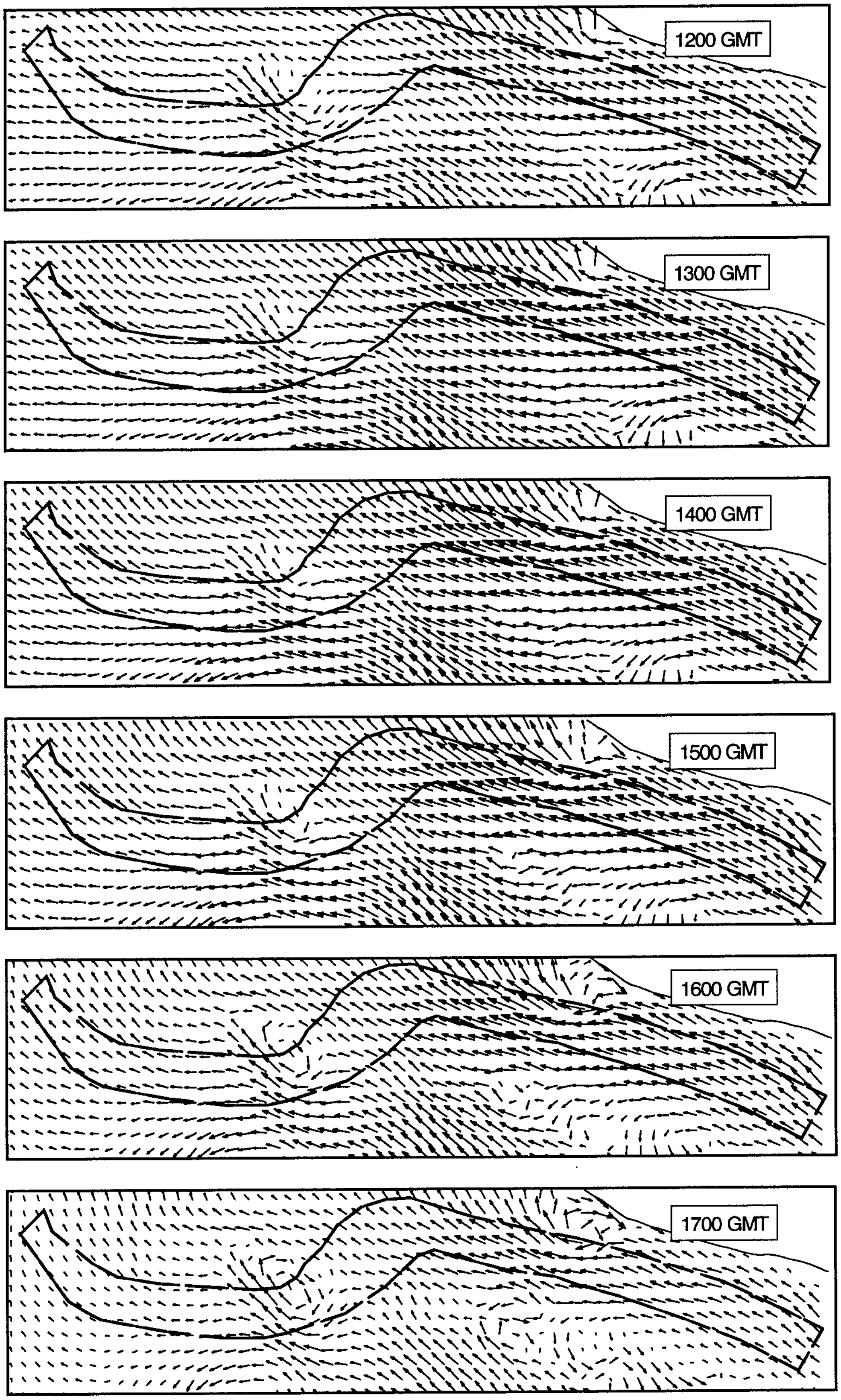


Figure G-18. Current patterns in Alternative 1 channel, September 1998 fair weather (Continued)

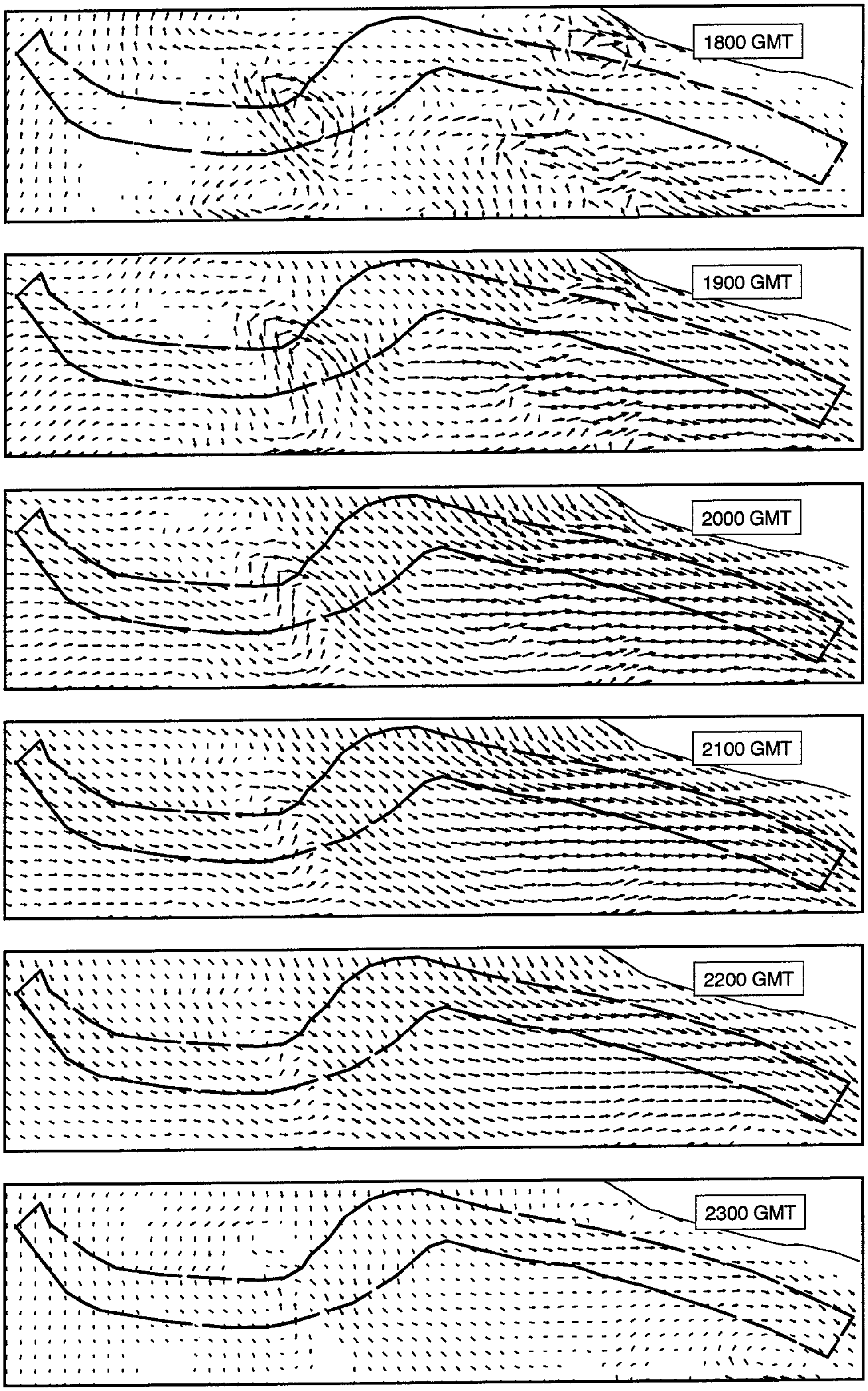


Figure G-18. (Concluded)

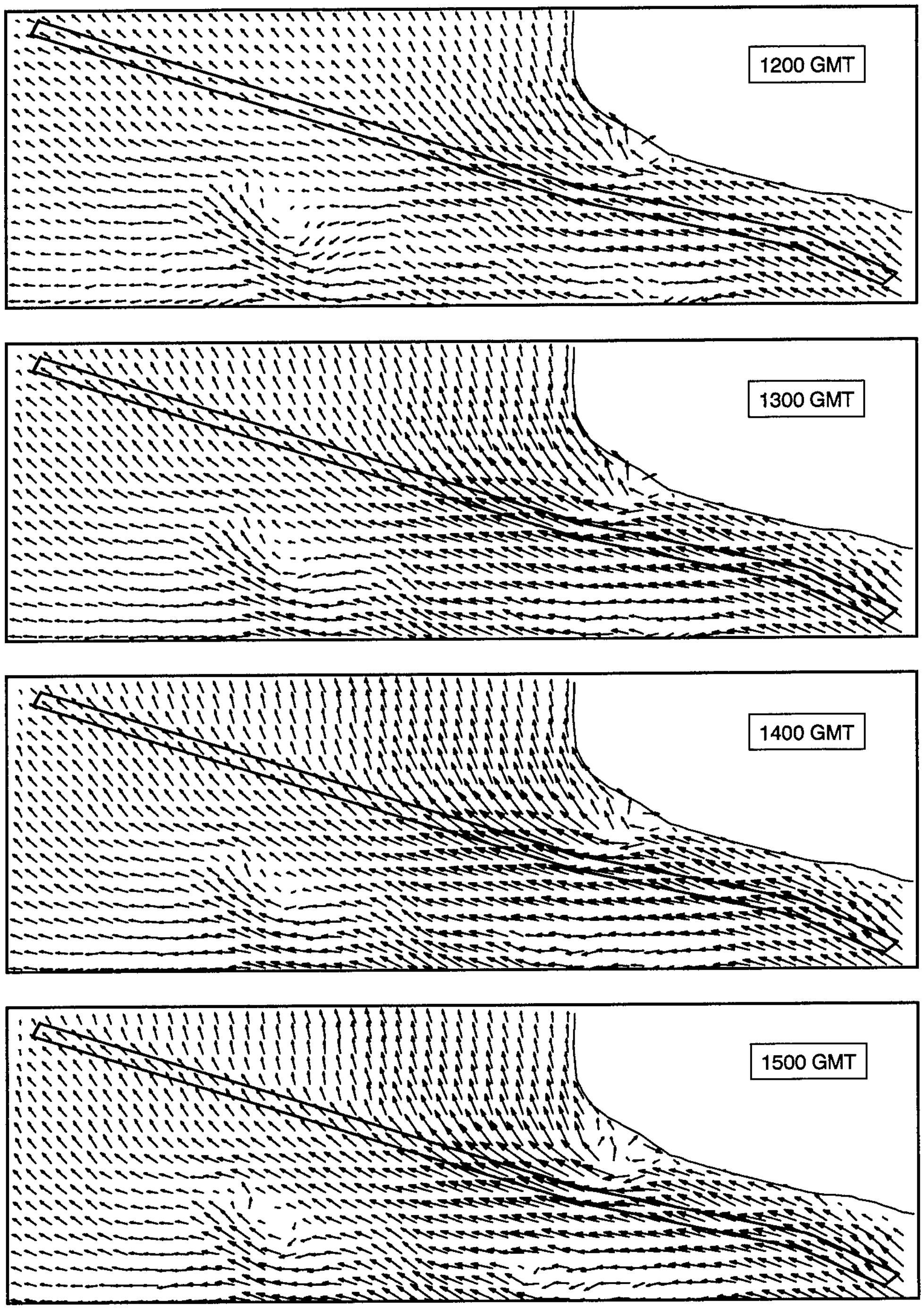


Figure G-19. Current patterns in Alternative 3A channel, September 1998 fair weather (Sheet 1 of 3)

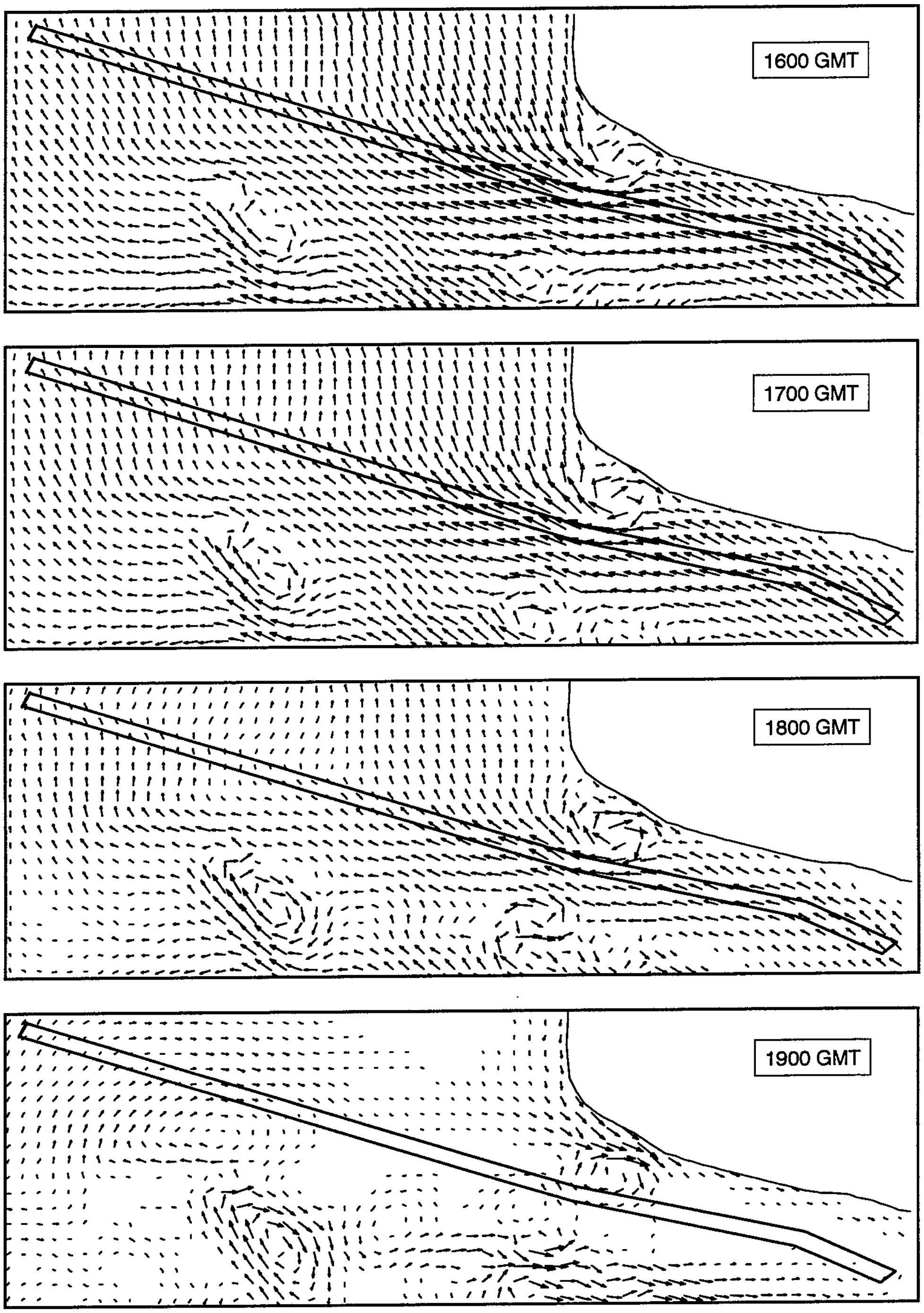


Figure G-19. (Sheet 2 of 3)

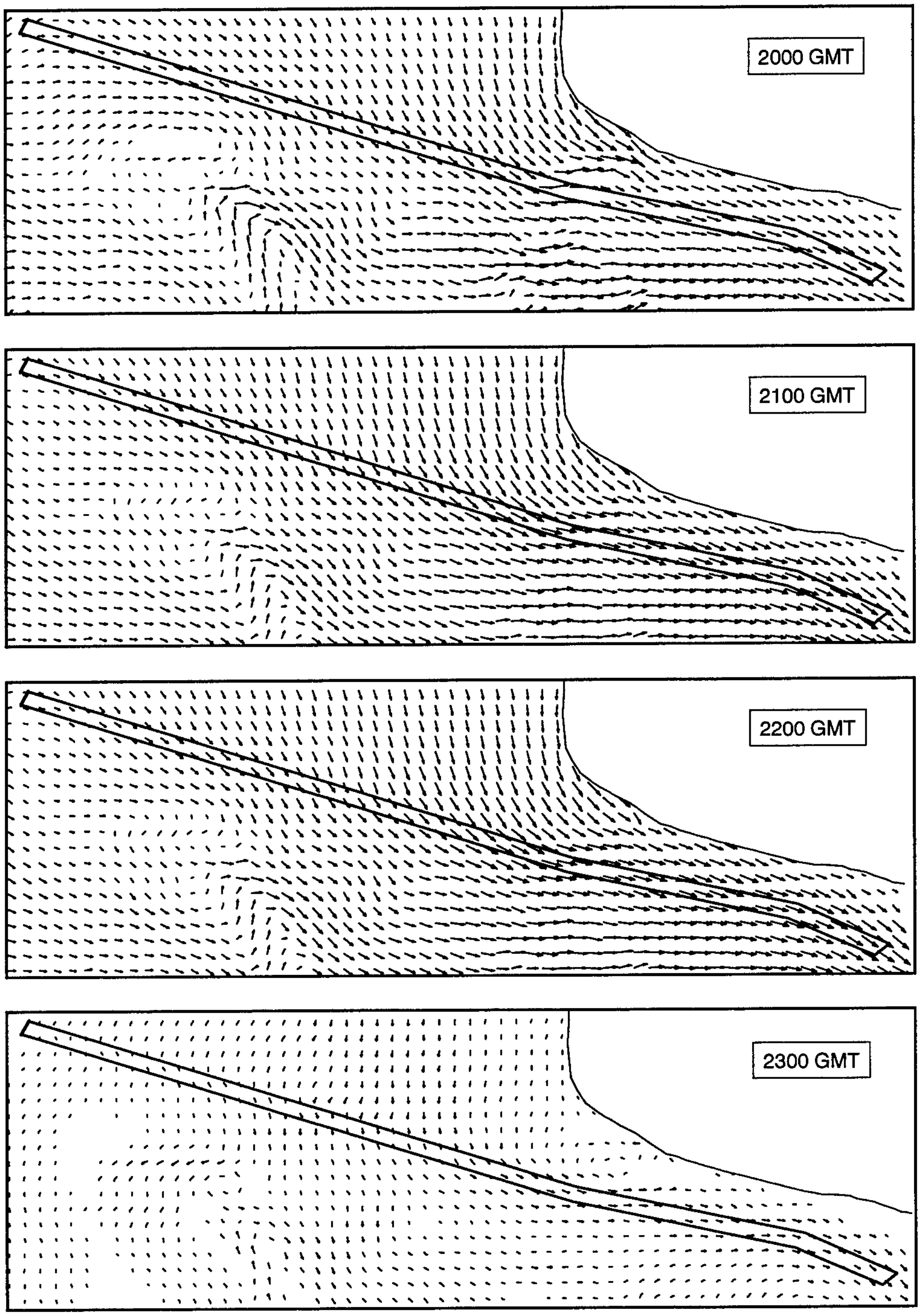


Figure G-19. (Sheet 3 of 3)

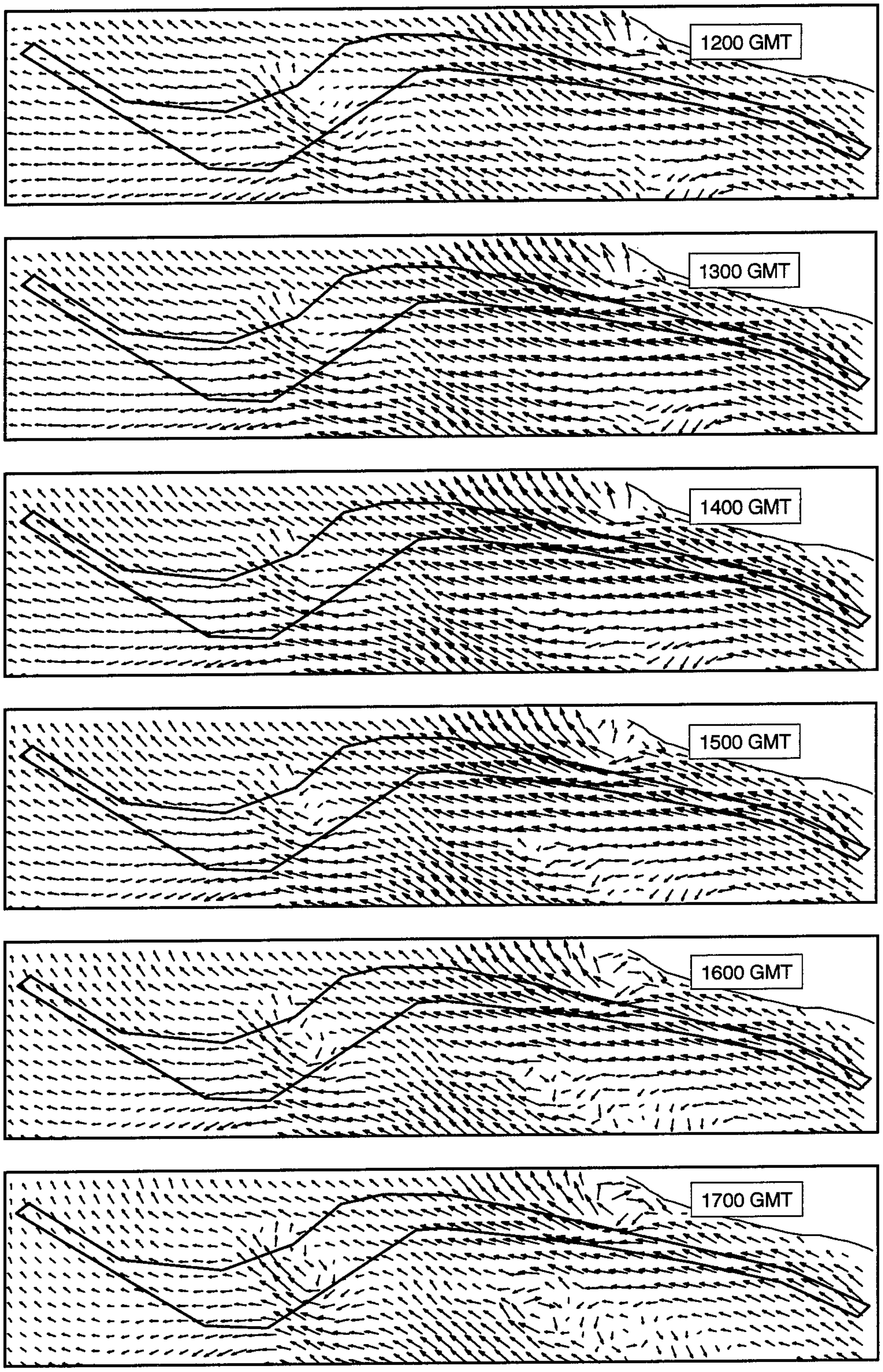


Figure G-20. Current patterns in Alternative 3B channel, September 1998 fair weather (Continued)

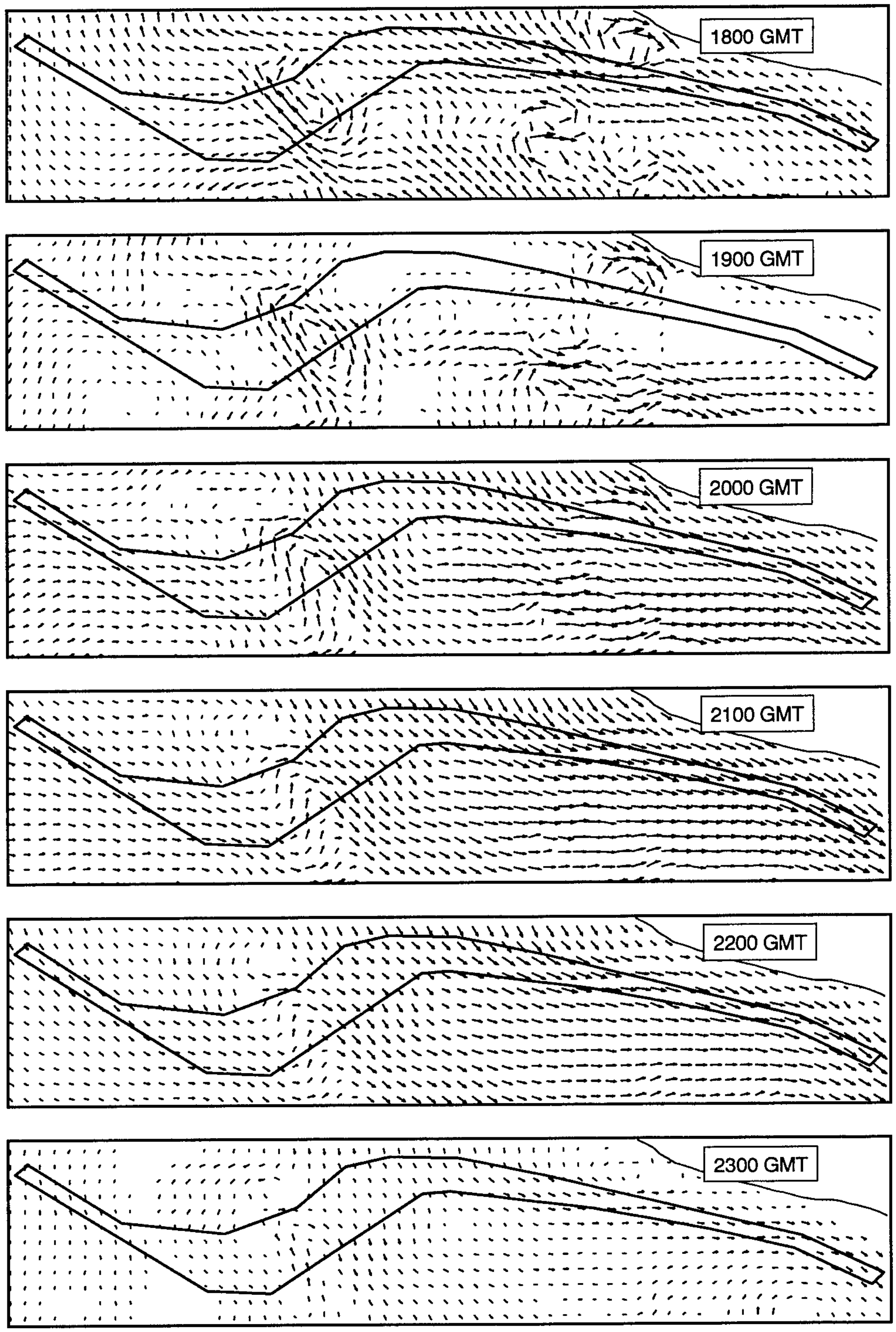


Figure G-20. (Concluded)

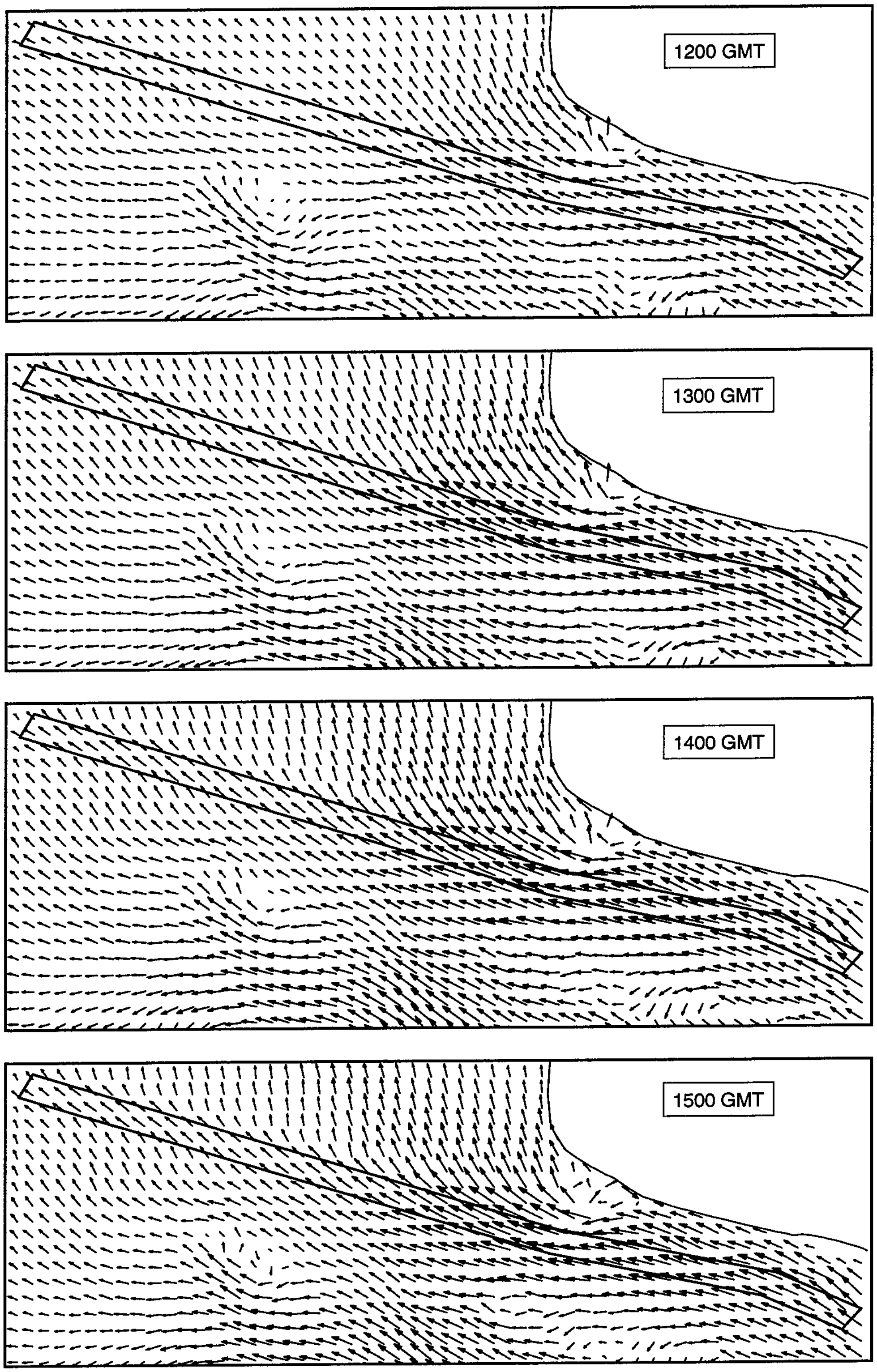


Figure G-21. Current patterns in Alternative 3F channel, September 1998 fair weather (Sheet 1 of 3)

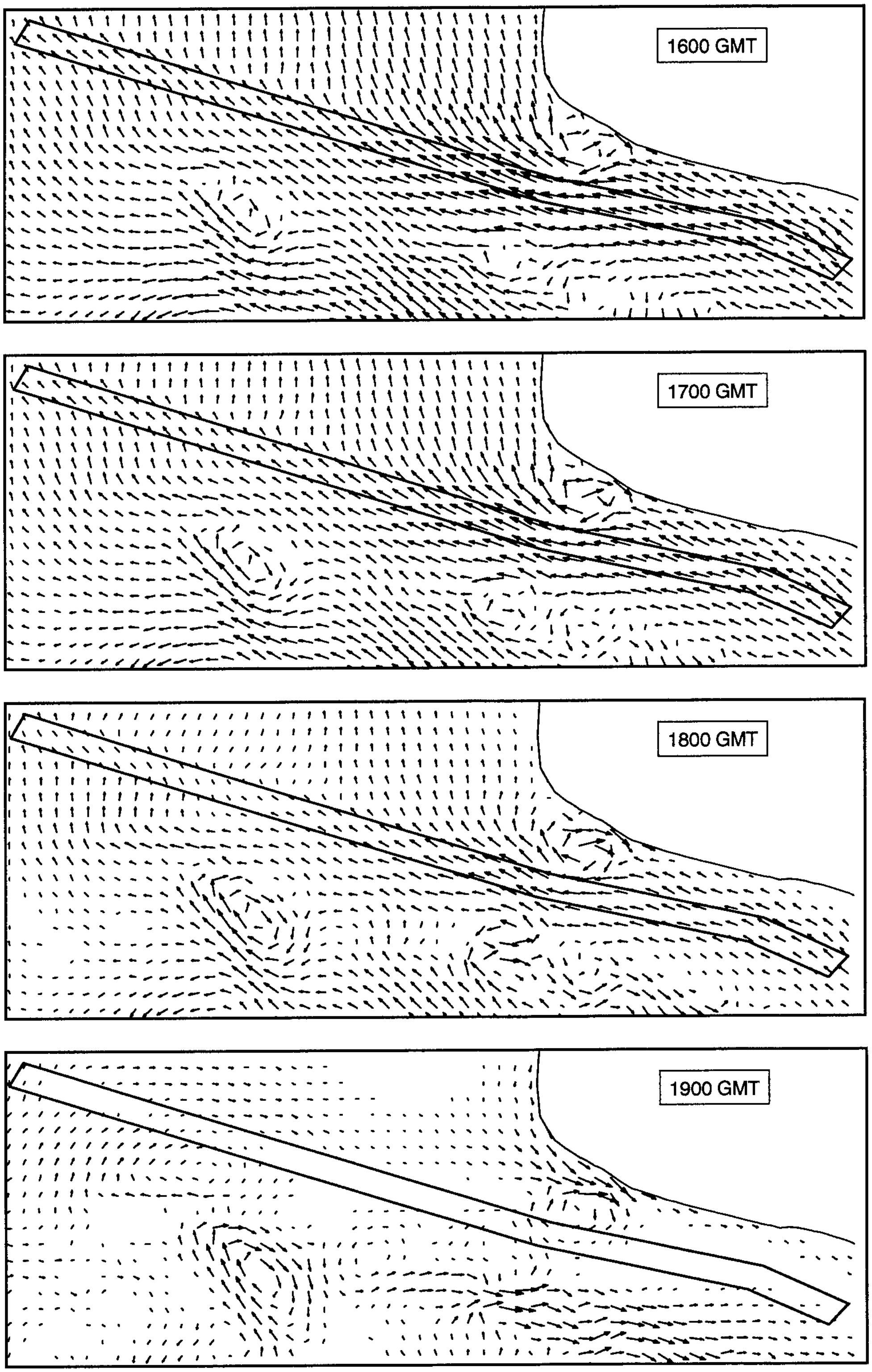


Figure G-21. (Sheet 2 of 3)

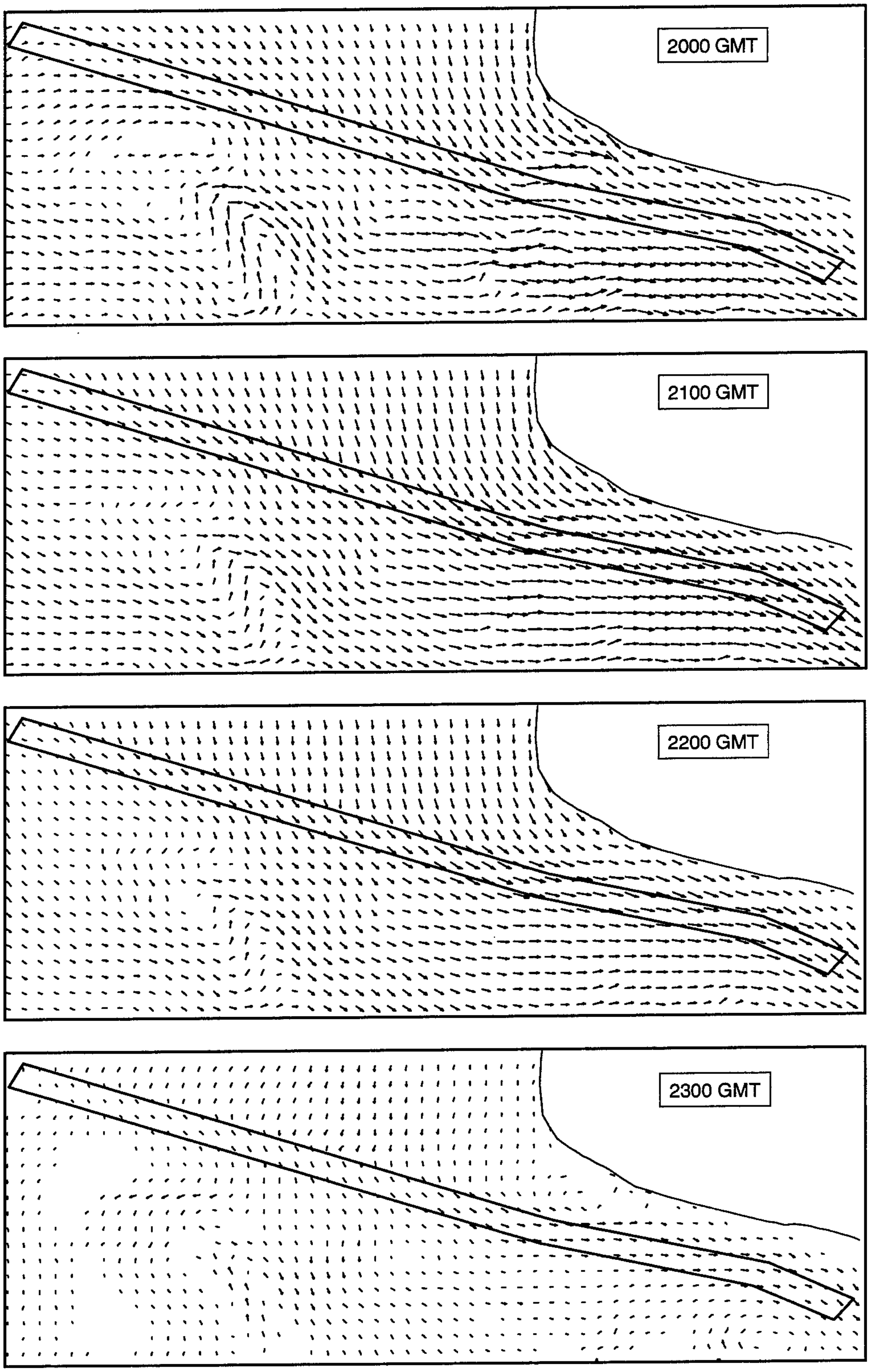


Figure G-21. (Sheet 3 of 3)

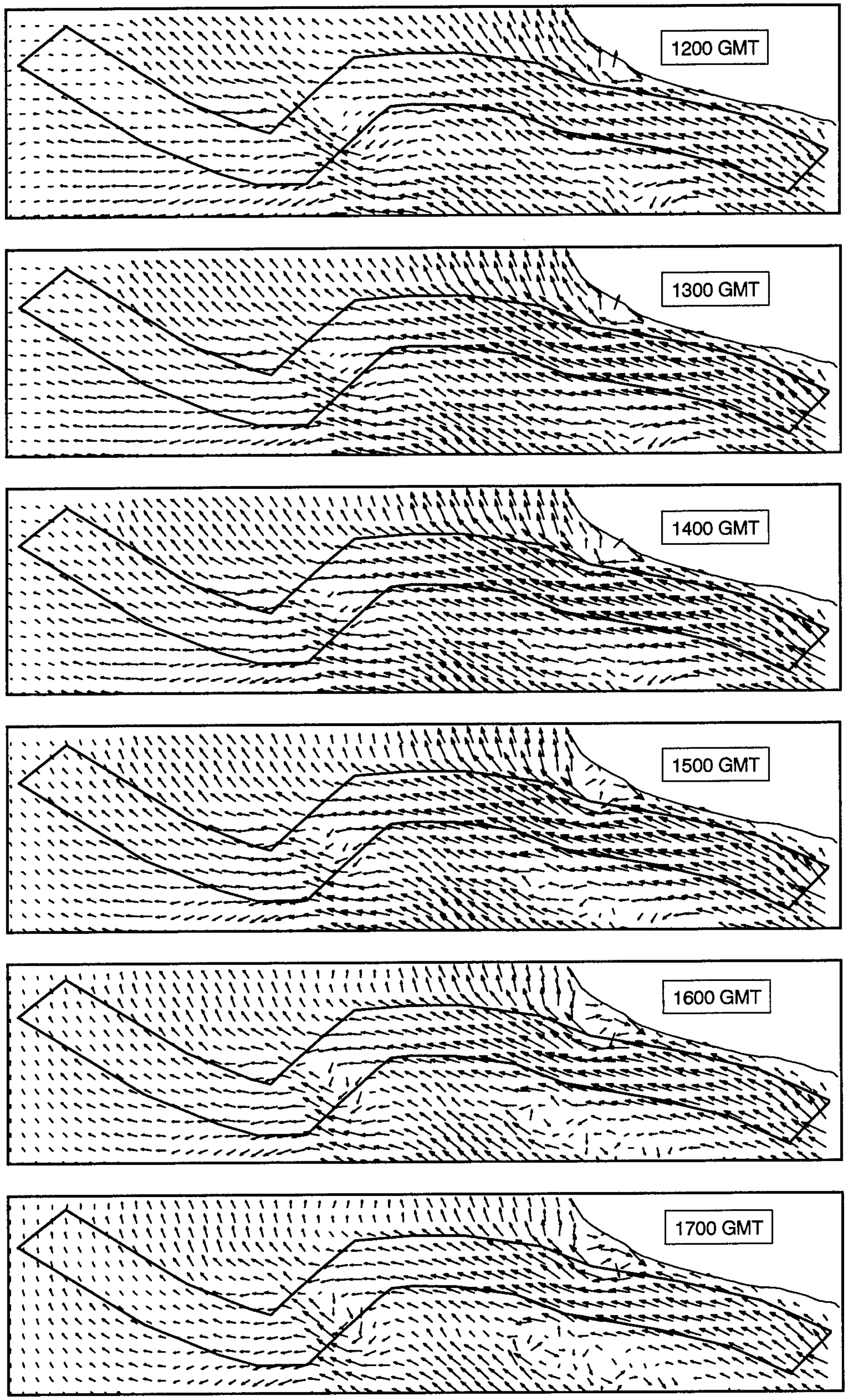
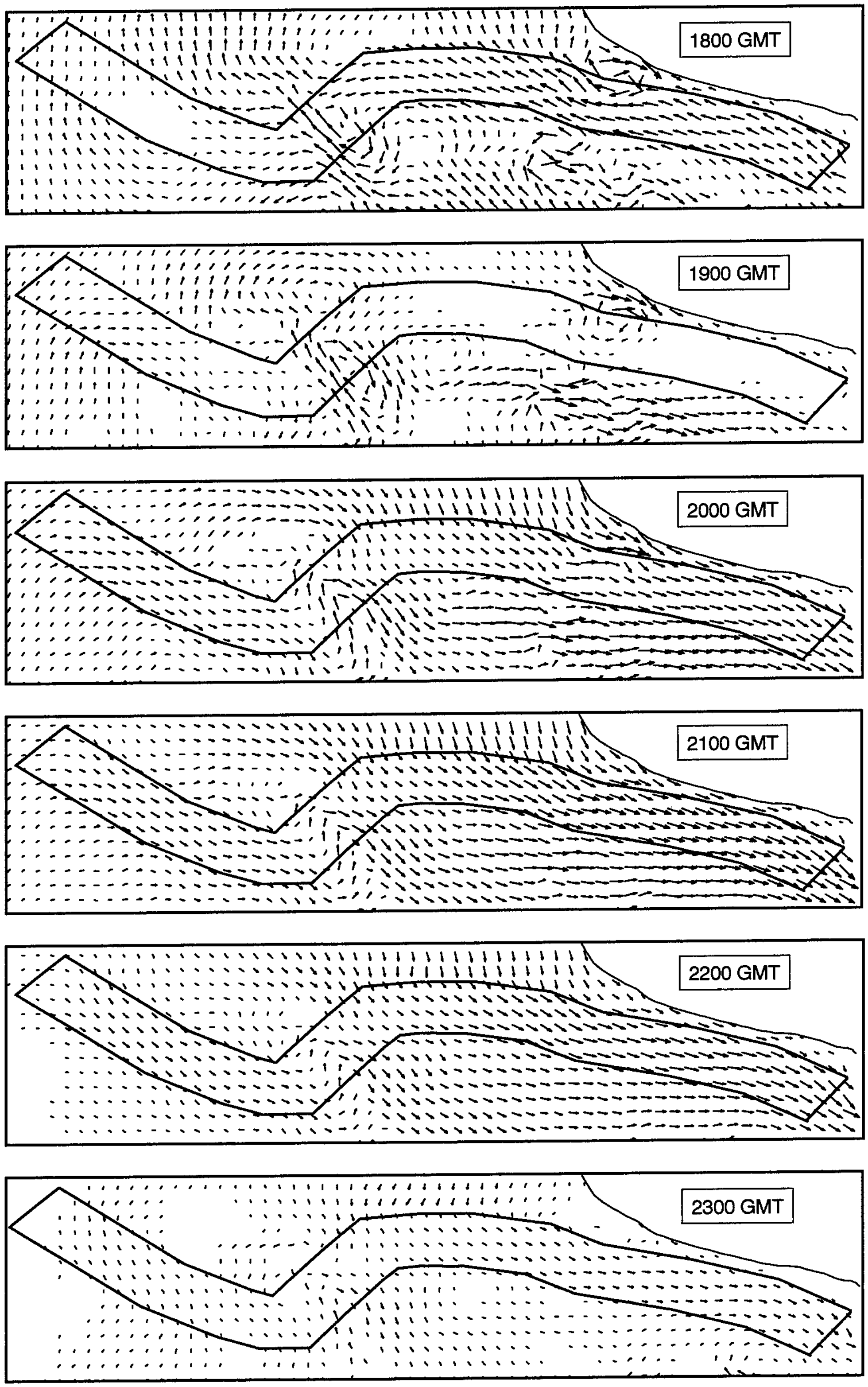


Figure G-22. Current patterns in Alternative 3G channel, September 1998 fair weather (Continued)



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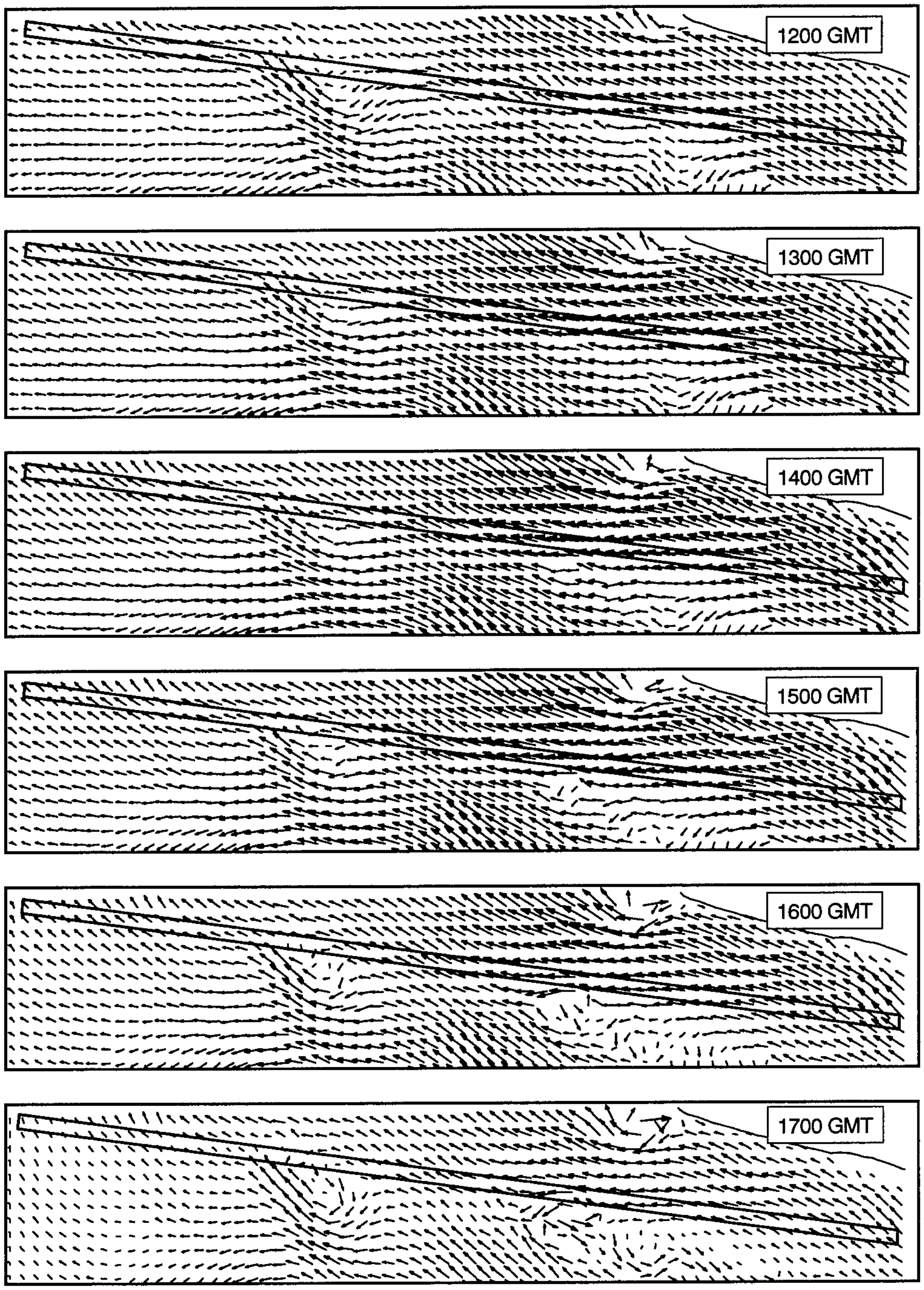


Figure G-23. Current patterns in Alternative 3H-a channel, September 1998 fair weather (Continued)

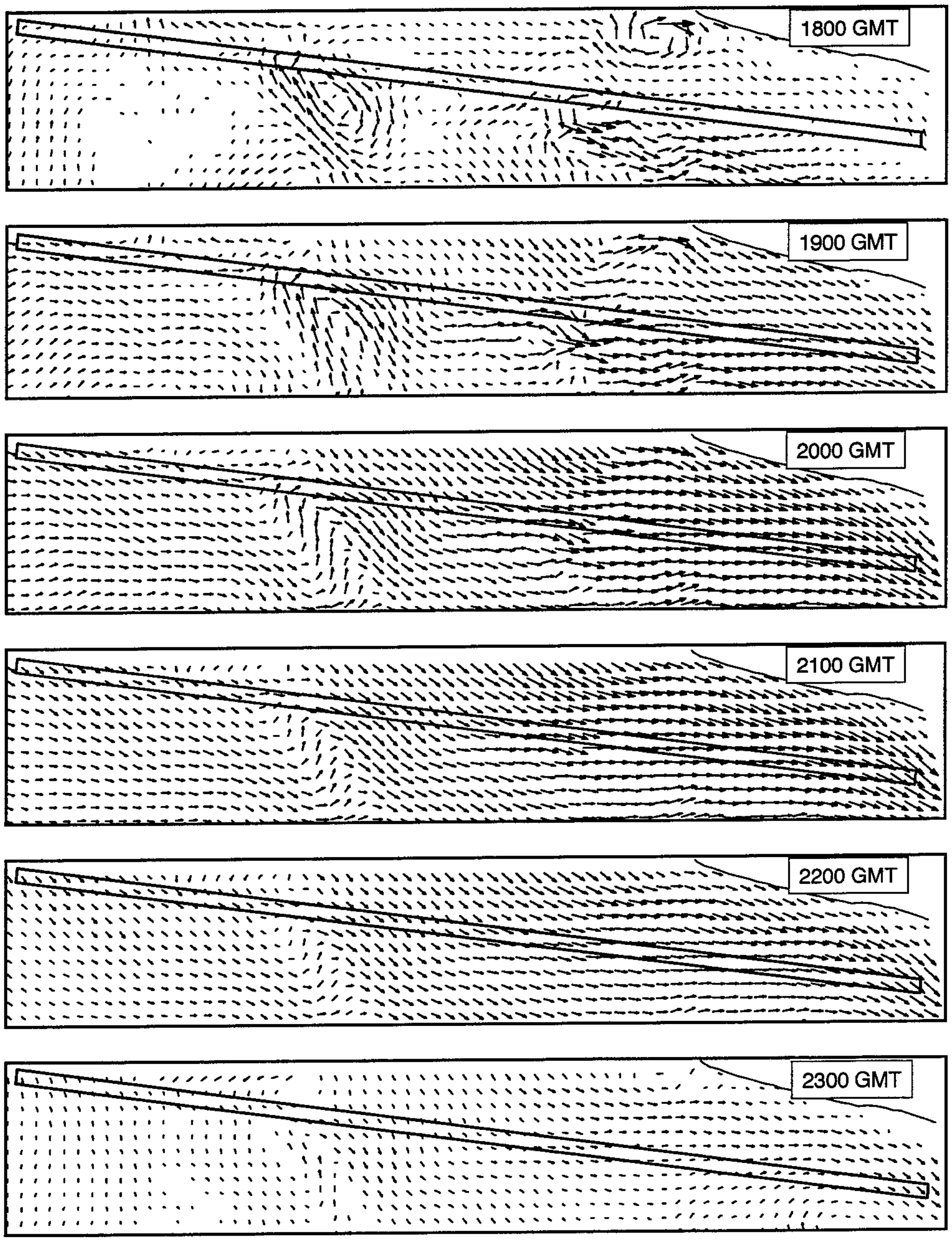


Figure G-23. (Concluded)

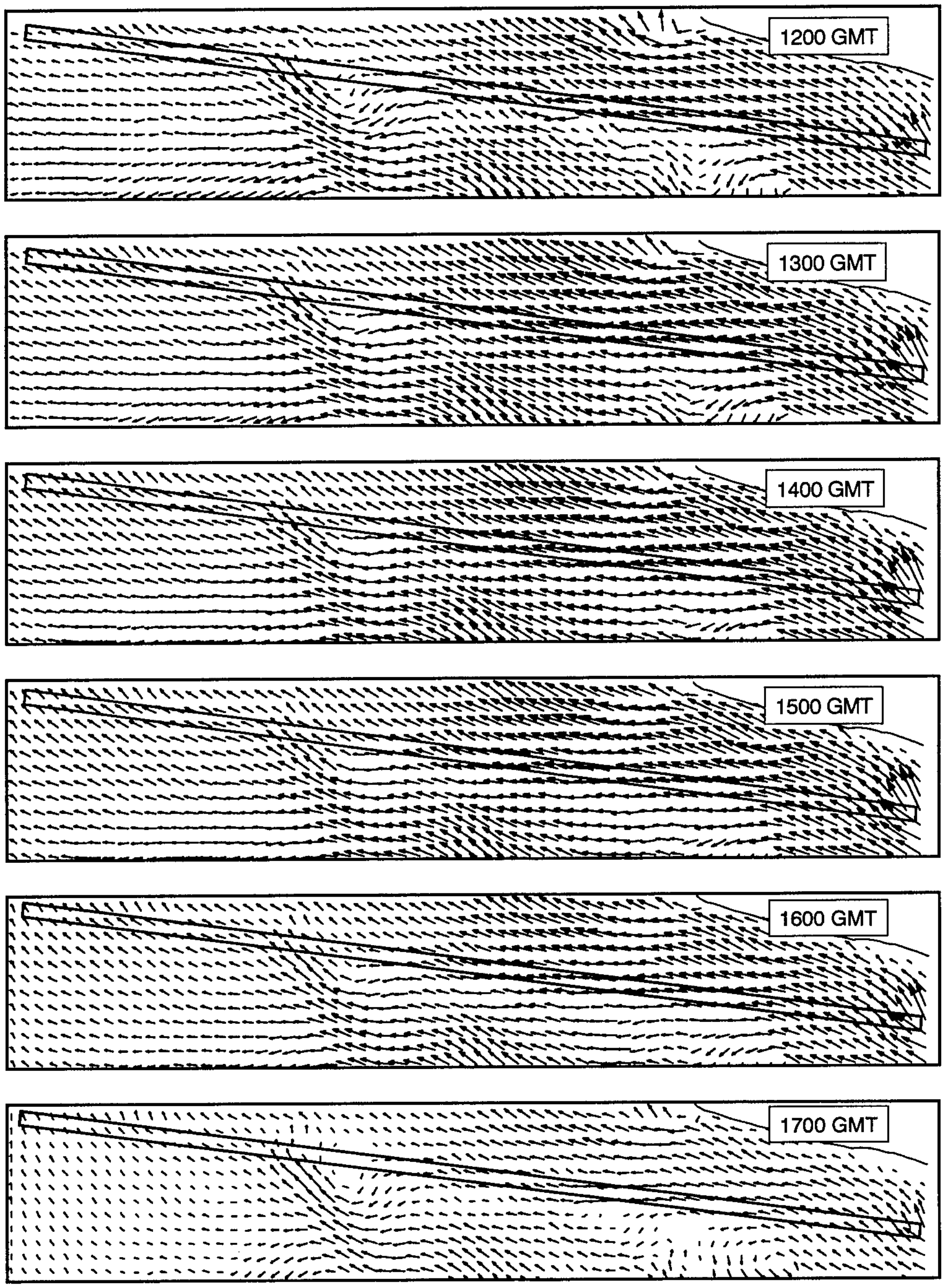
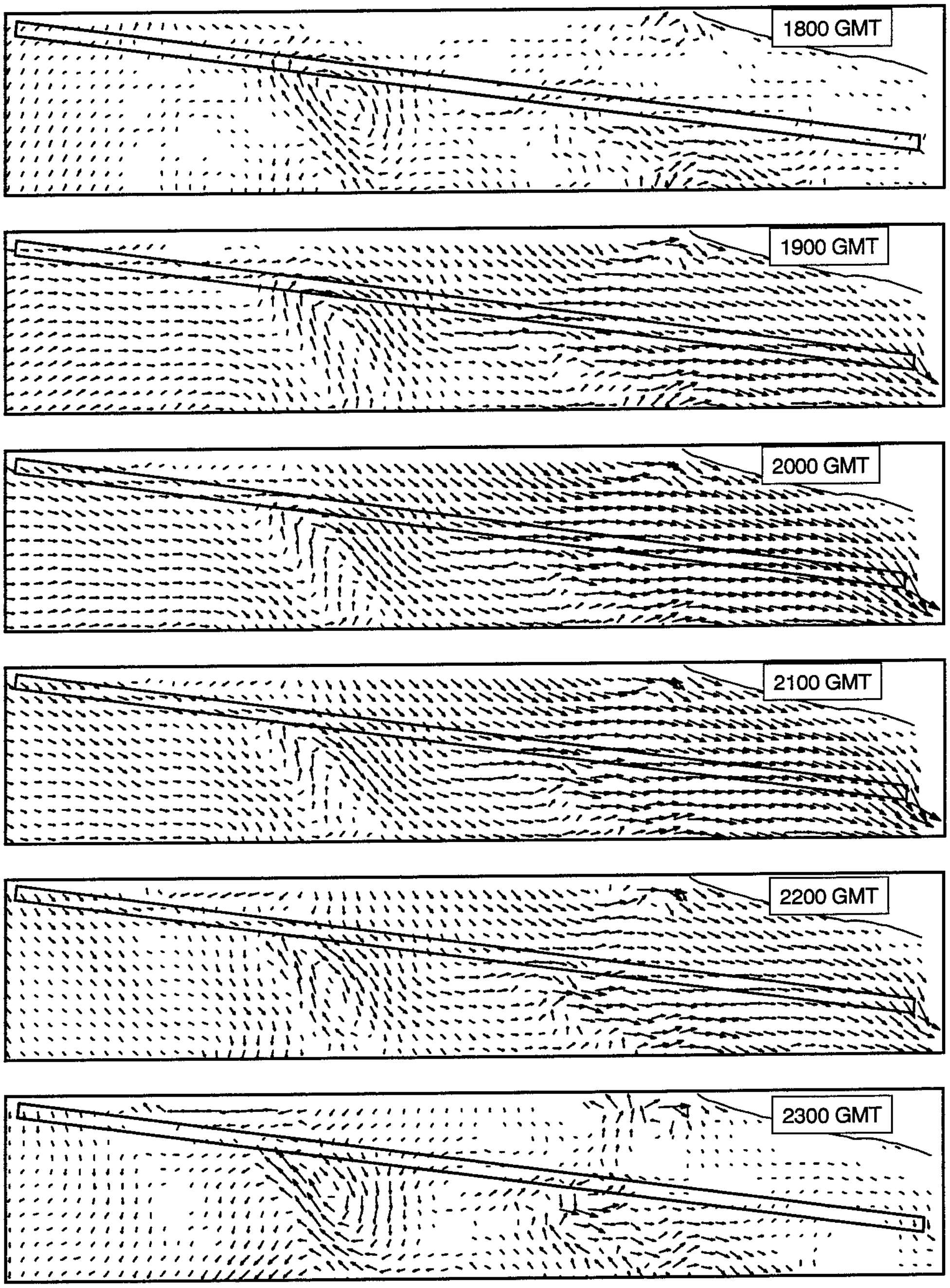


Figure G-24. Current patterns in Alternative 3H-b channel, September 1998 fair weather (Continued)



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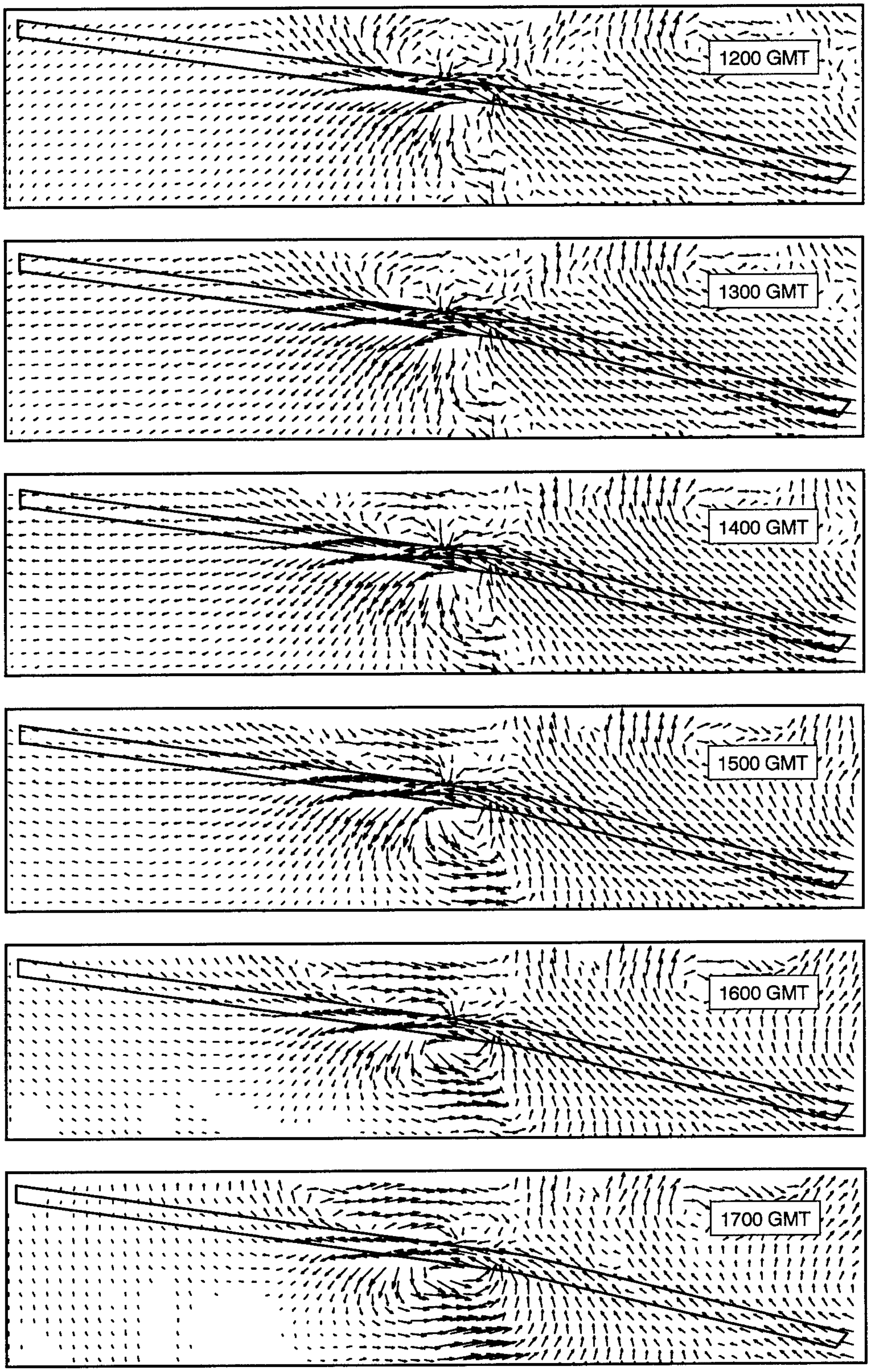

Figure G-25. Current patterns in Alternative 4A channel, September 1998 fair weather (Continued)

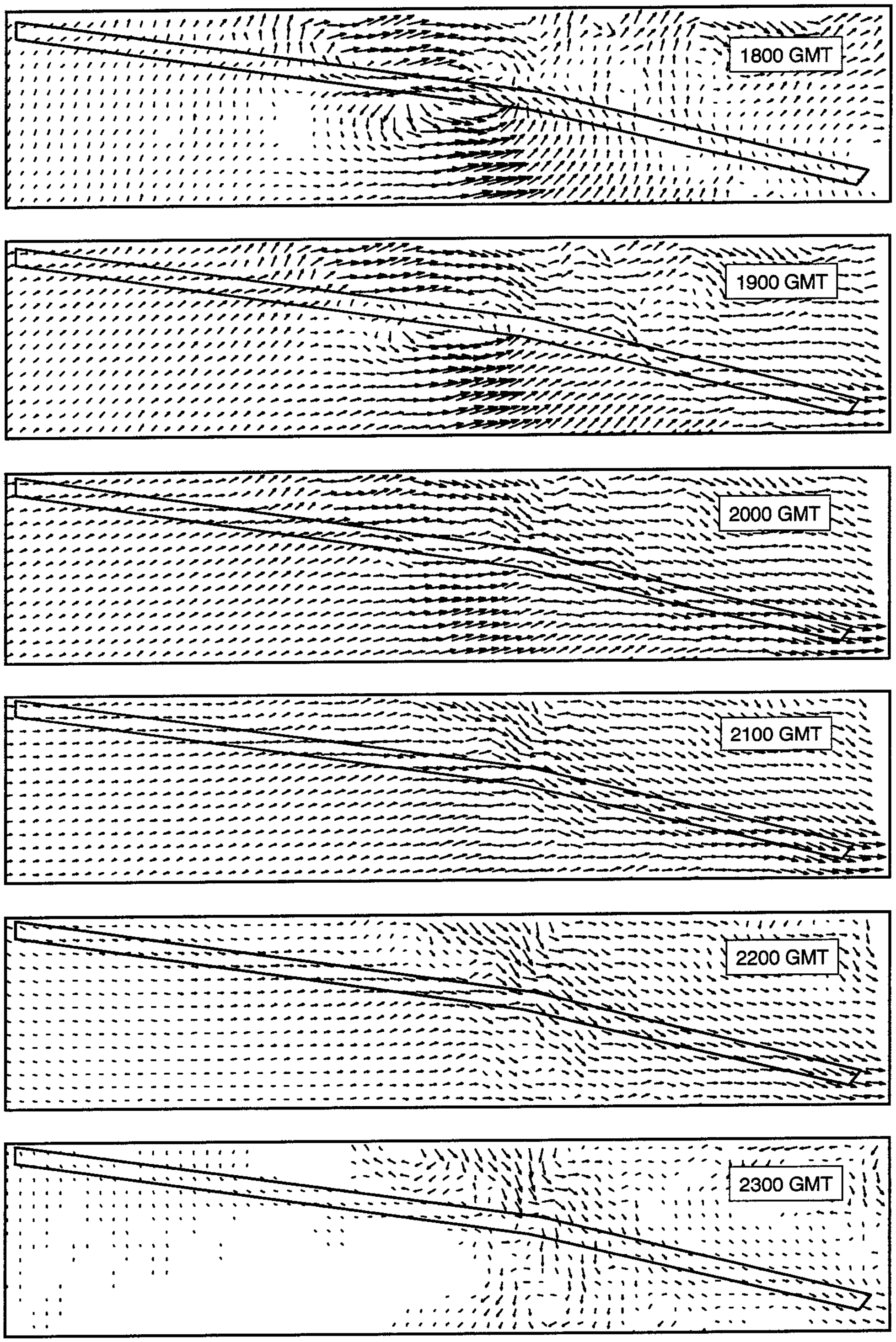

Figure G-25. (Concluded)

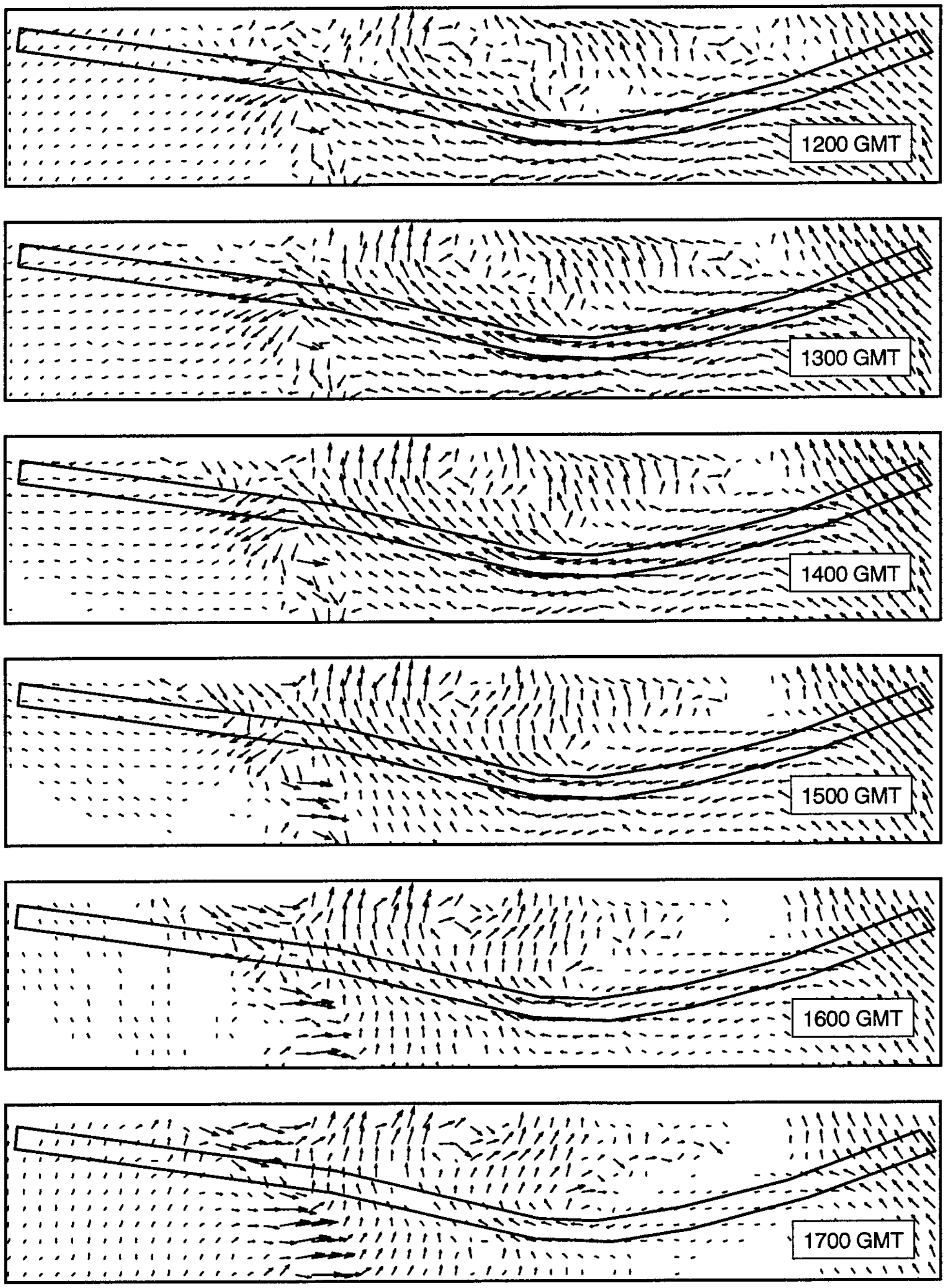

Figure G-26. Current patterns in Alternative 4E channel, September 1998 fair weather (Continued)

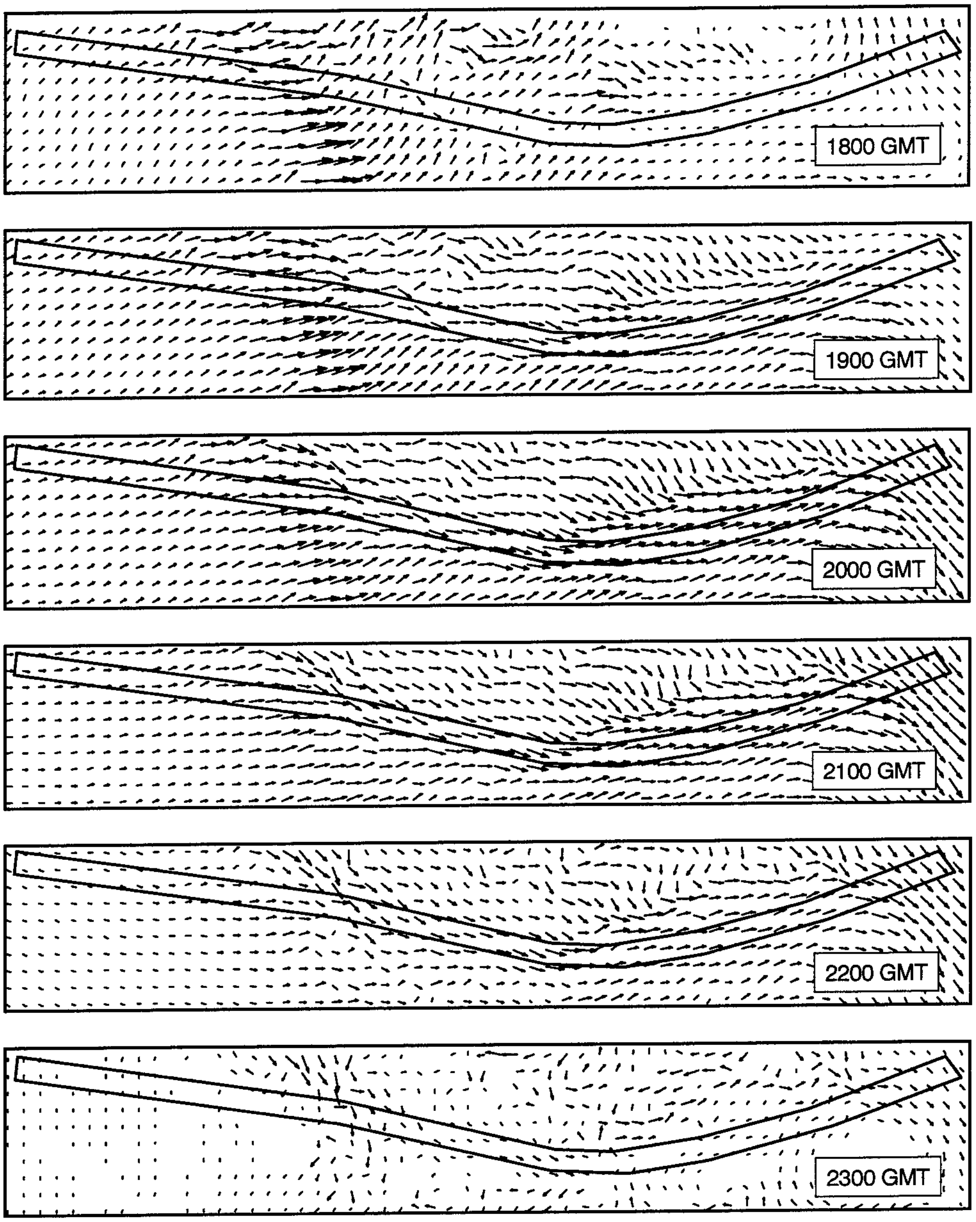

Figure G-26. (Concluded)

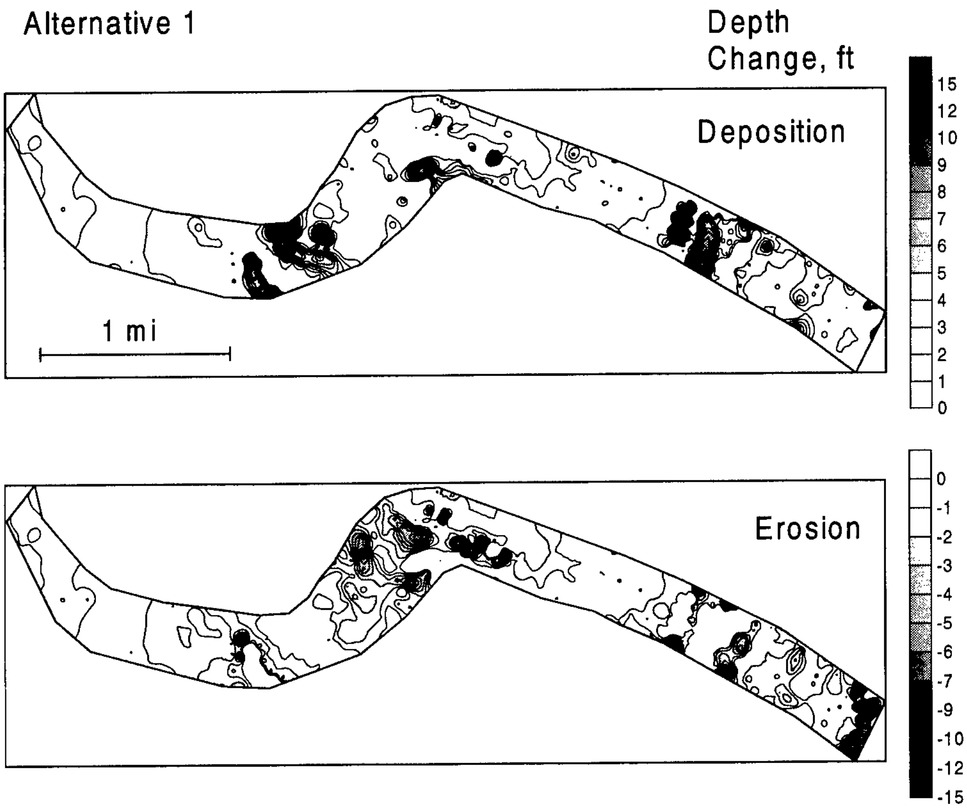

Figure G-27. Calculated deposition and erosion in Alternative 1 channel, January 1998 storm

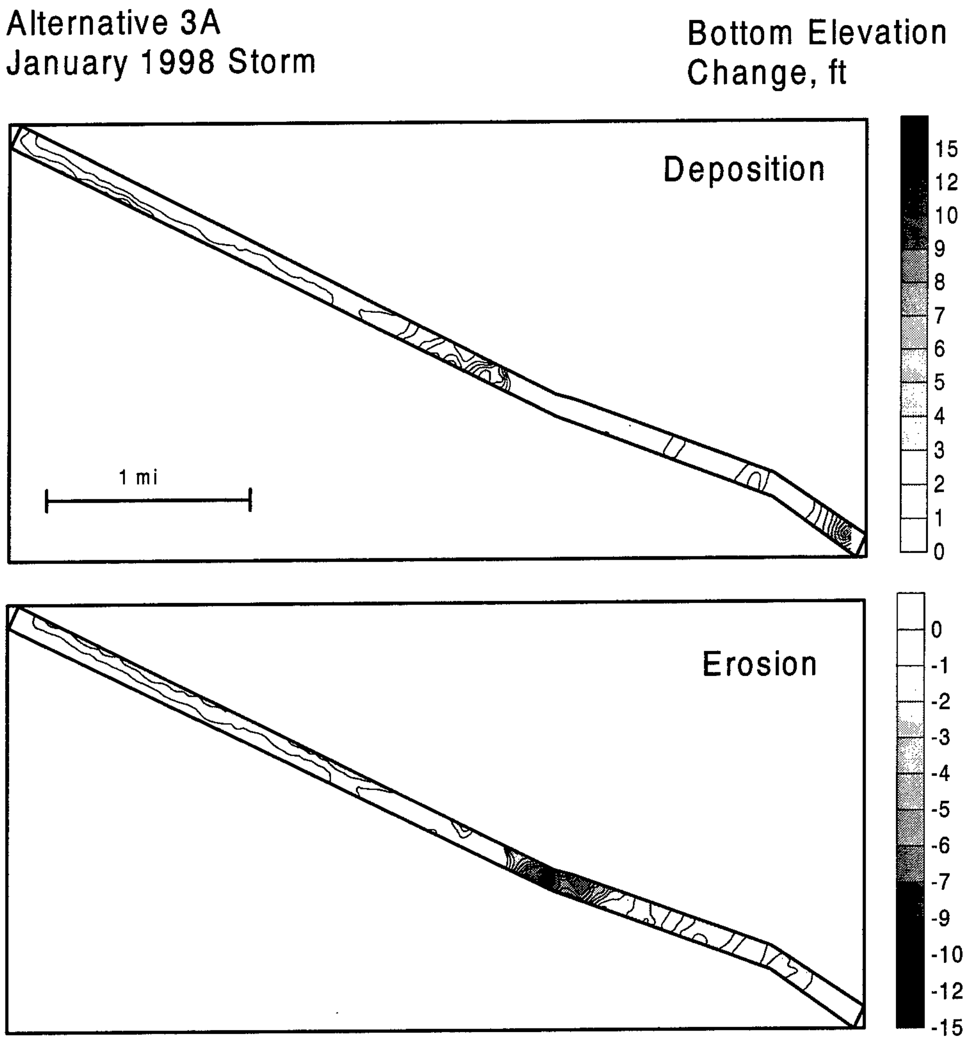

Figure G-28. Calculated deposition and erosion in Alternative 3A channel, January 1998 storm

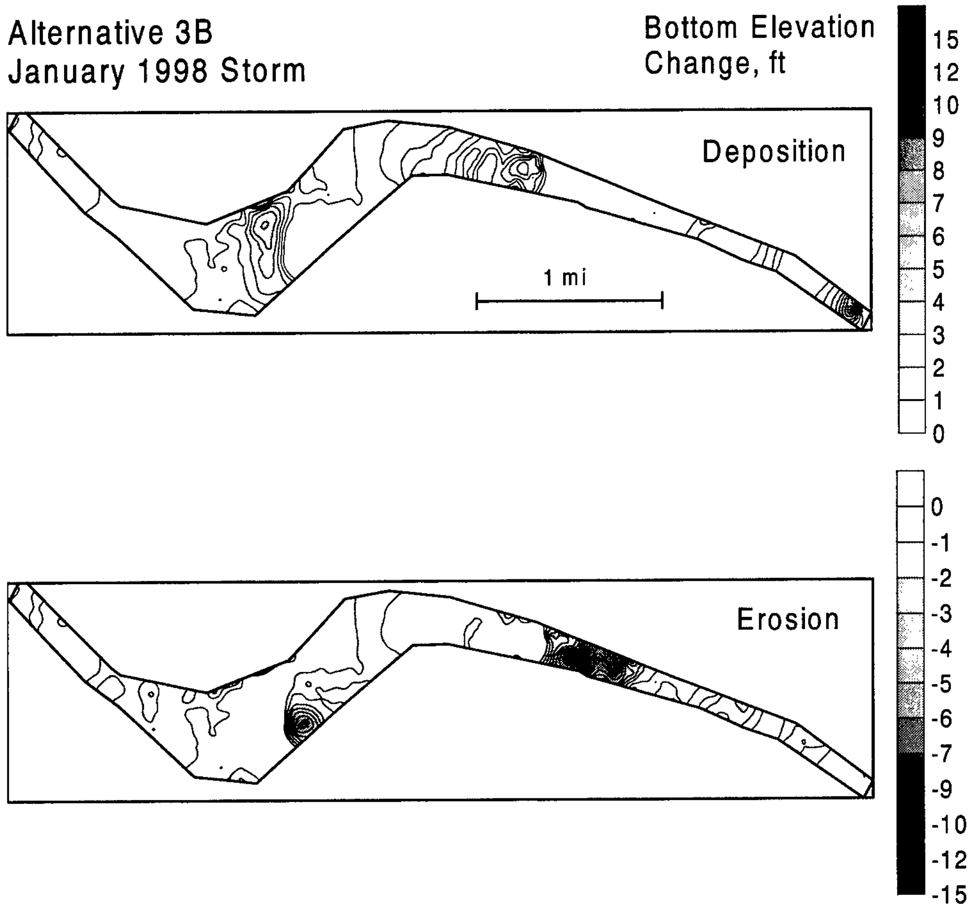

Figure G-29. Calculated deposition and erosion in Alternative 3B channel, January 1998 storm

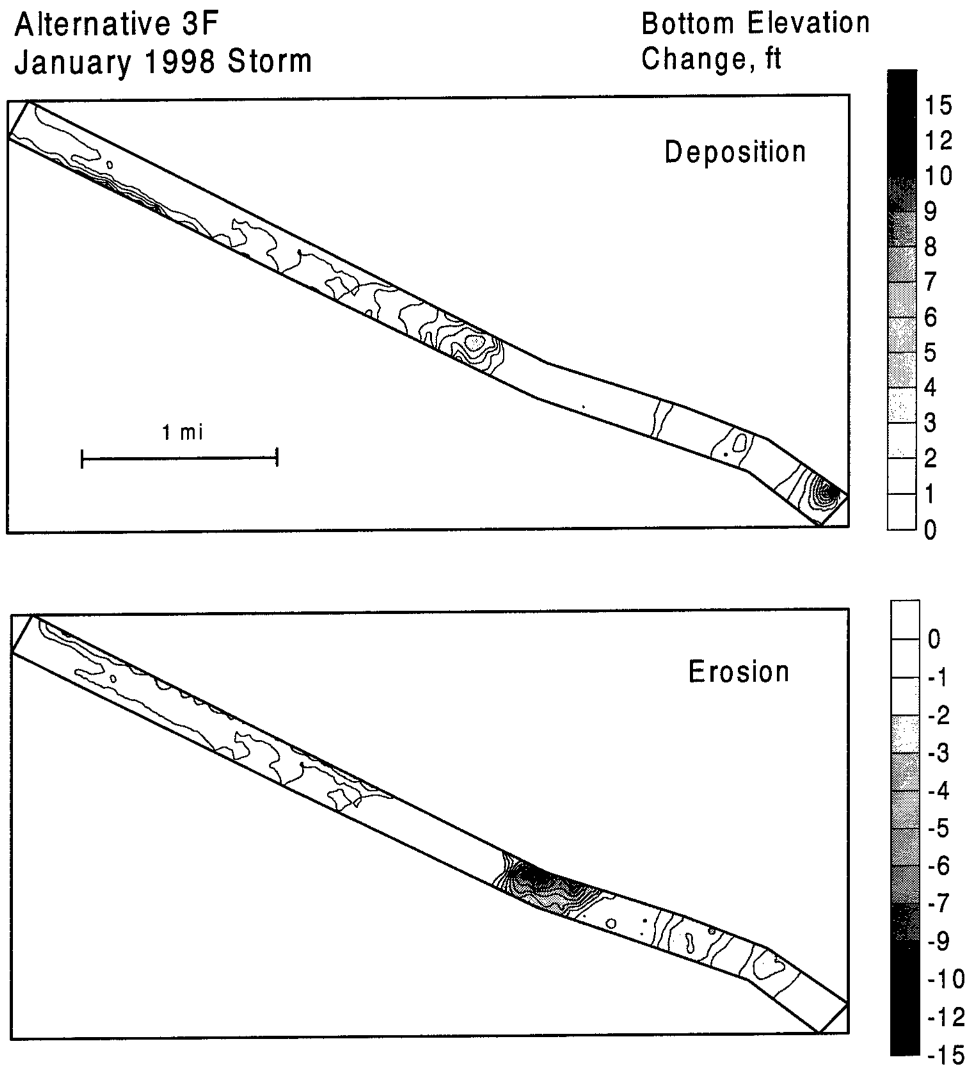

Figure G-30. Calculated deposition and erosion in Alternative 3F channel, January 1998 storm

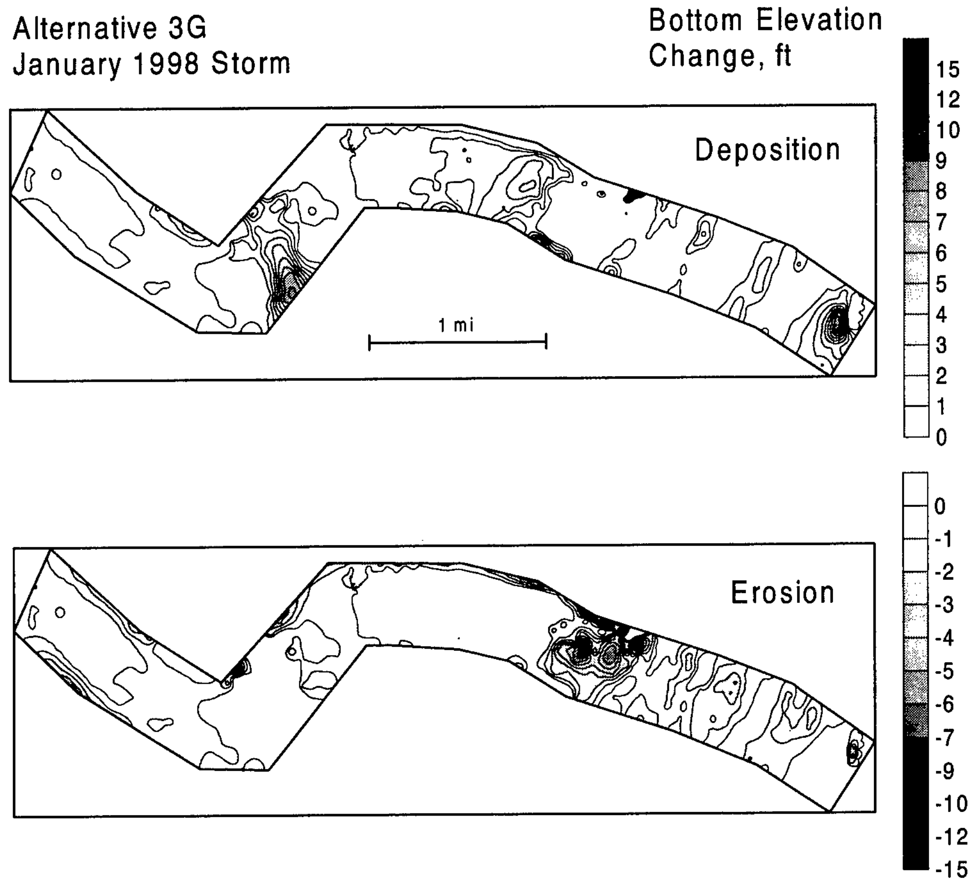

Figure G-31. Calculated deposition and erosion in Alternative 3G channel, January 1998 storm

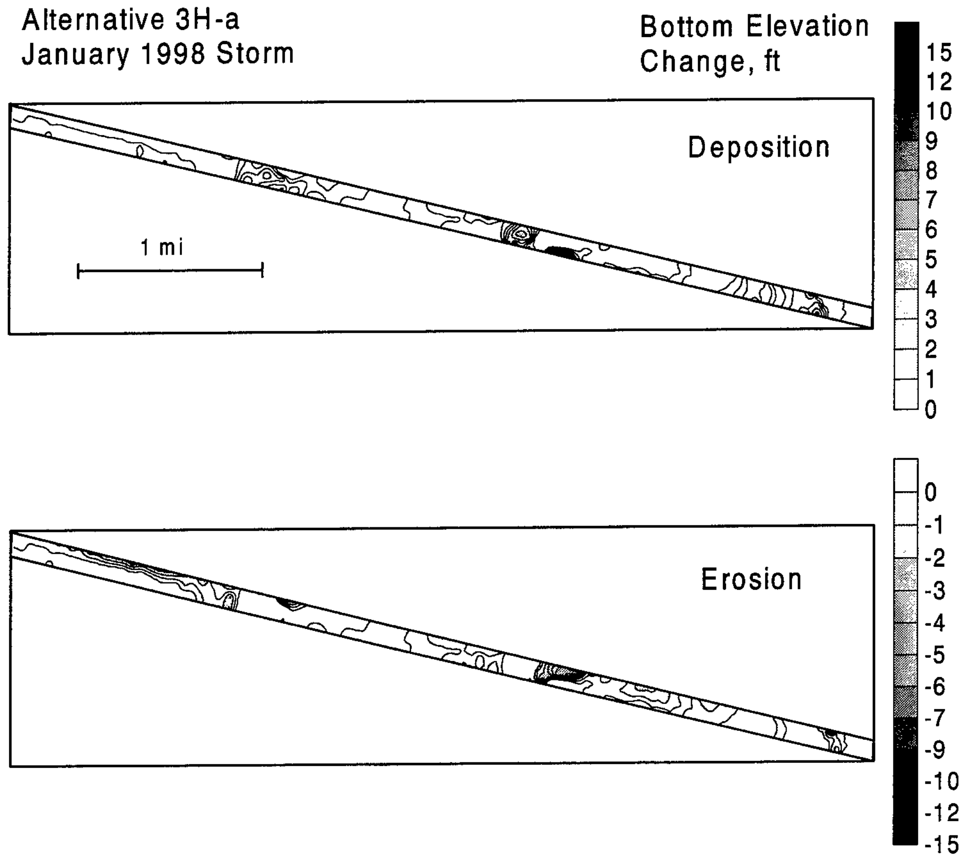

Figure G-32. Calculated deposition and erosion in Alternative 3H-a channel, January 1998 storm

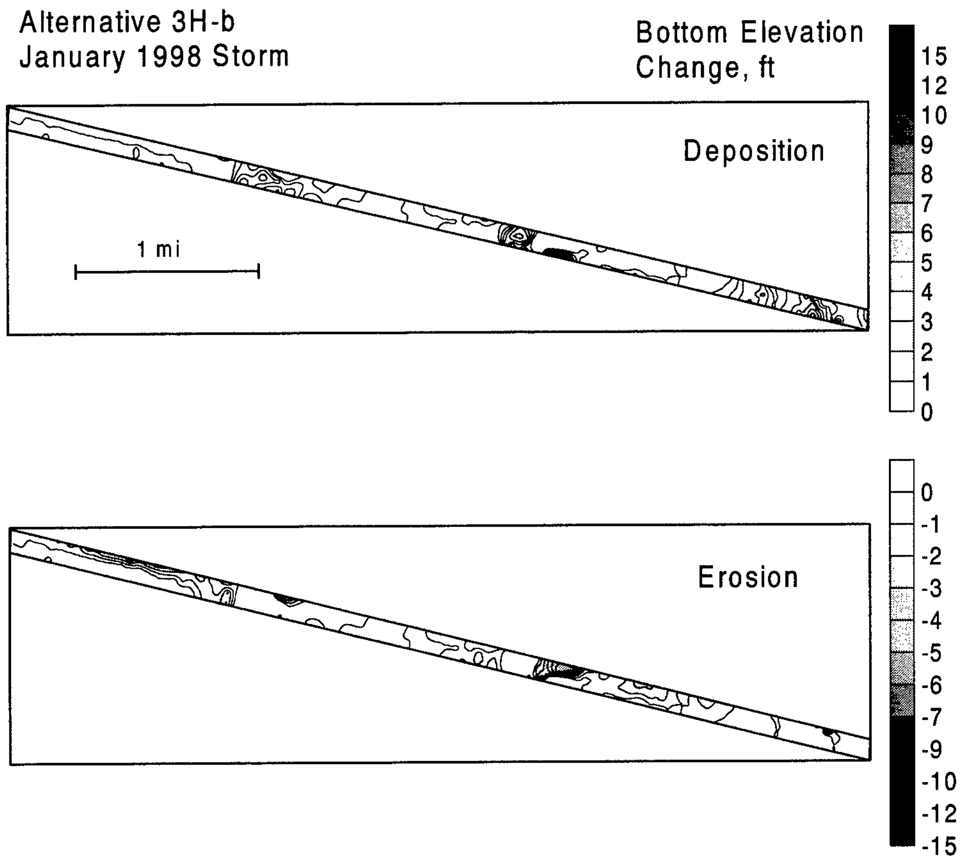

Figure G-33. Calculated deposition and erosion in Alternative 3H-b channel, January 1998 storm

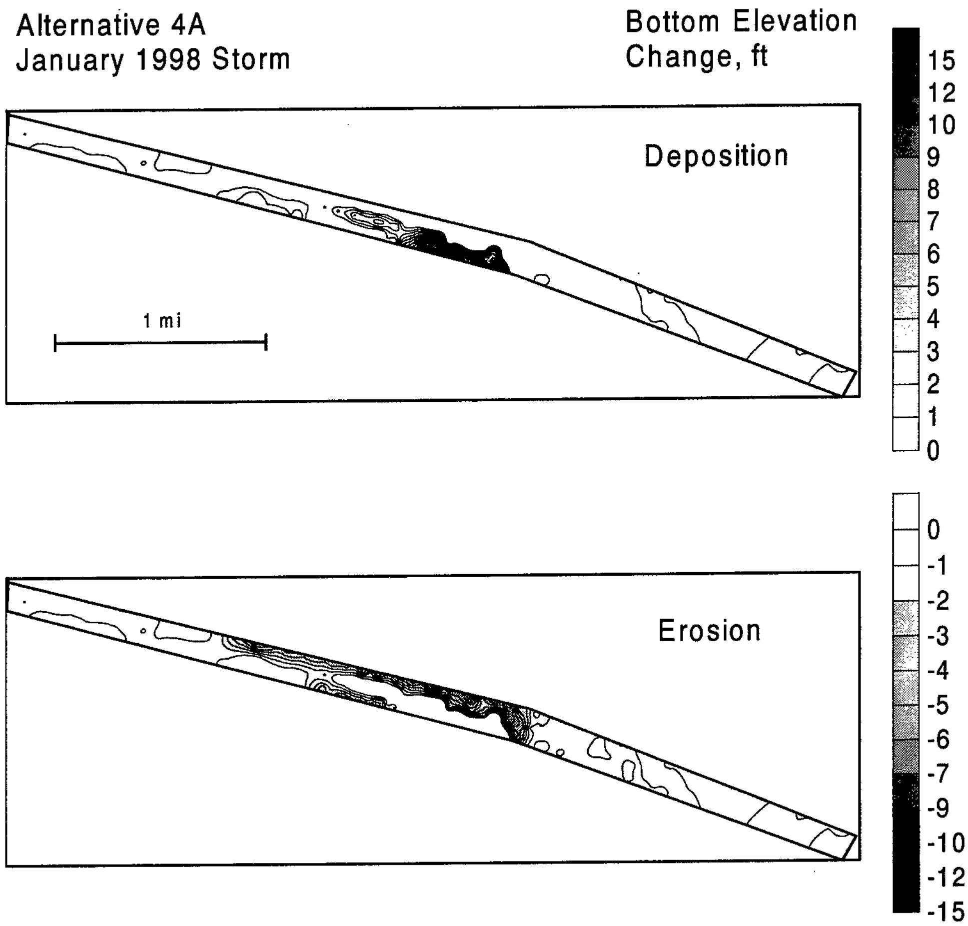

Figure G-34. Calculated deposition and erosion in Alternative 4A channel, January 1998 storm

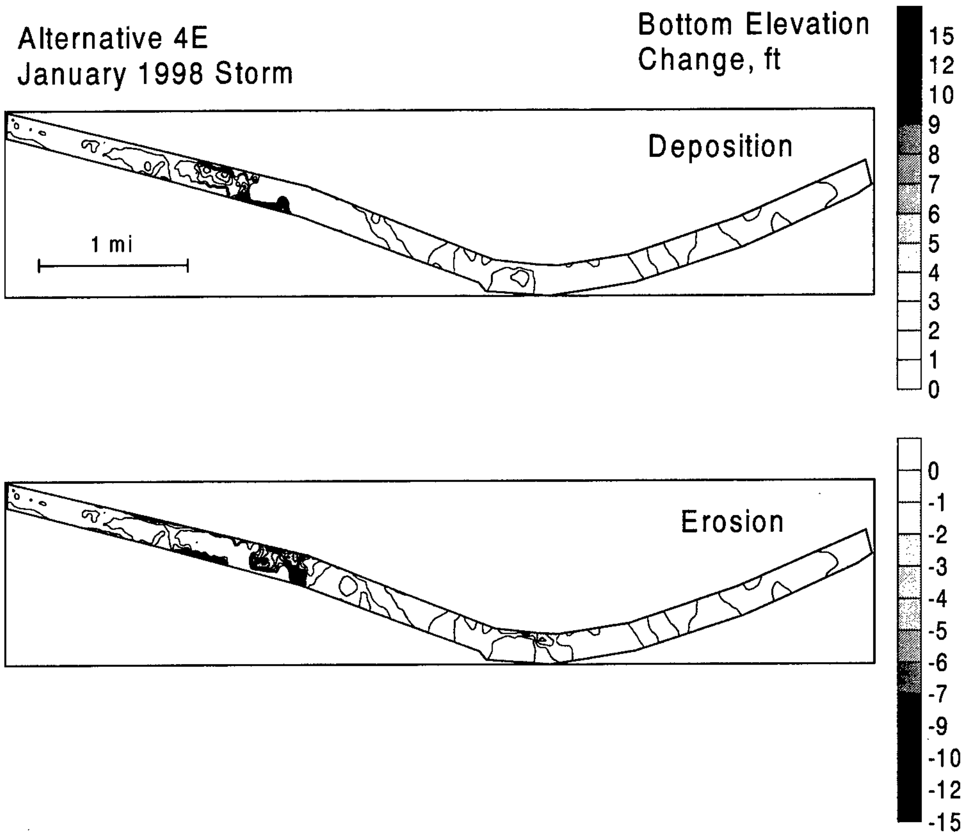

Figure G-35. Calculated deposition and erosion in Alternative 4E channel, January 1998 storm

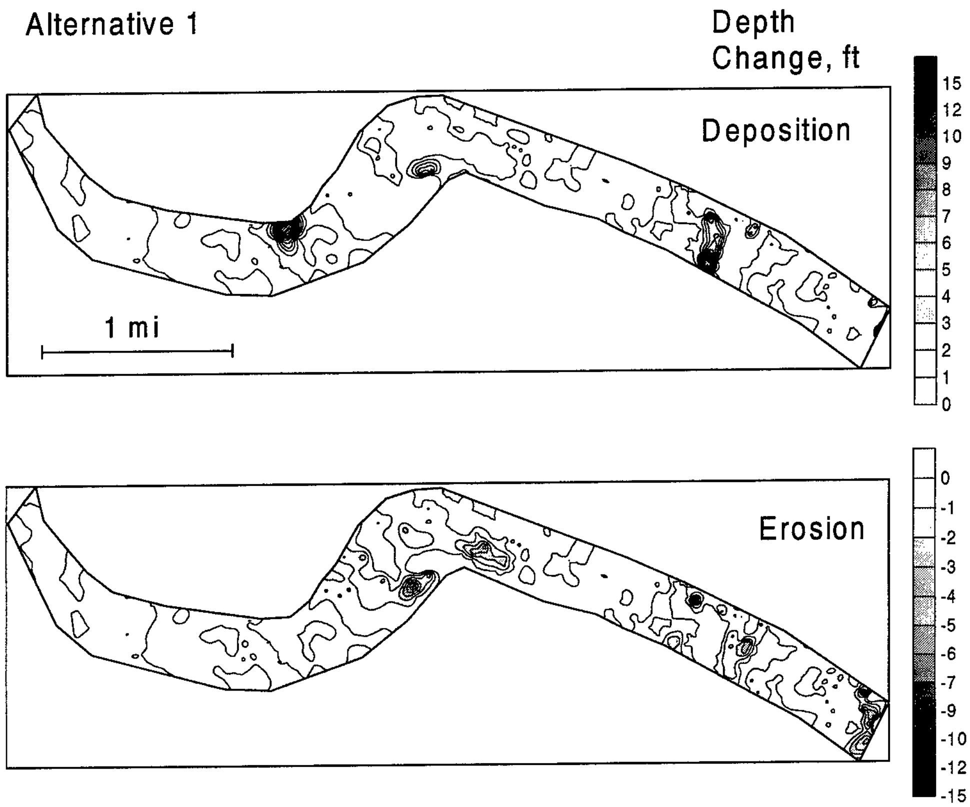

Figure G-36. Calculated deposition and erosion in Alternative 1 channel, September 1998 fair weather

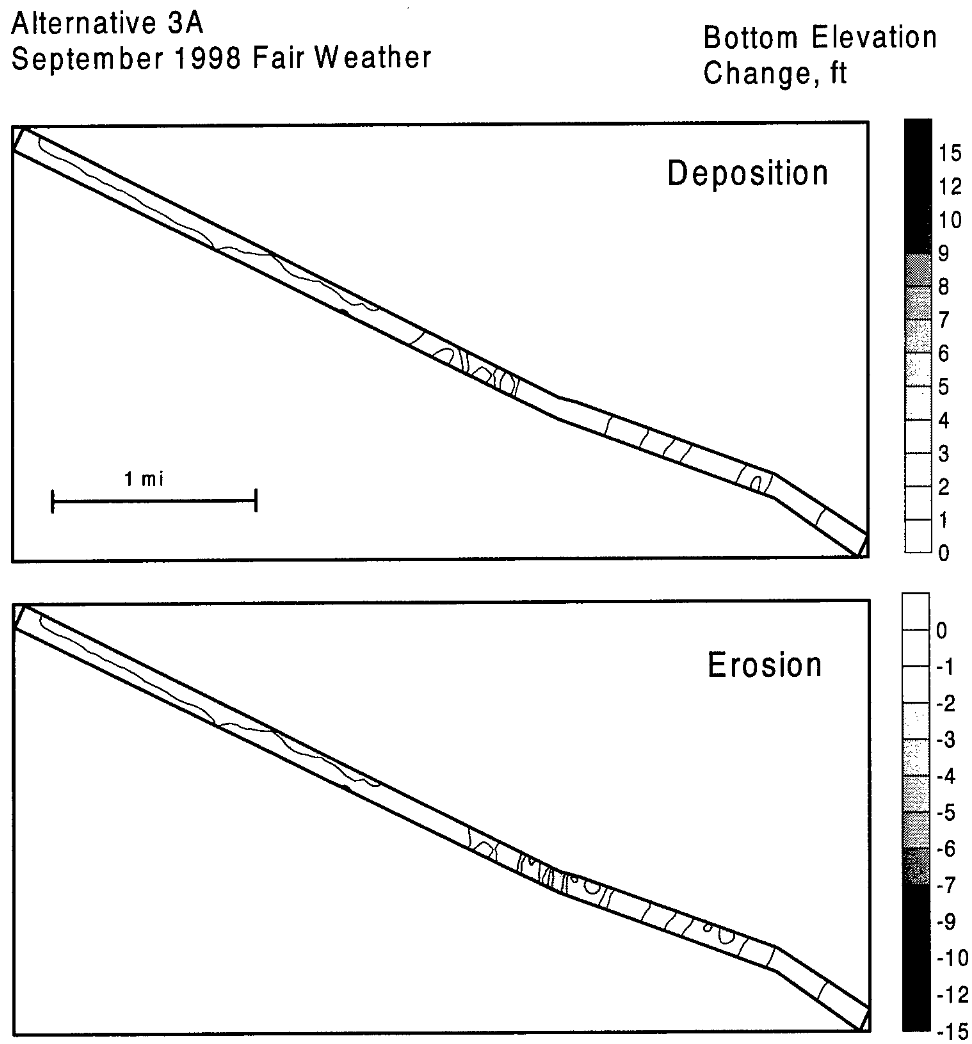

Figure G-37. Calculated deposition and erosion in Alternative 3A channel, September 1998 fair weather

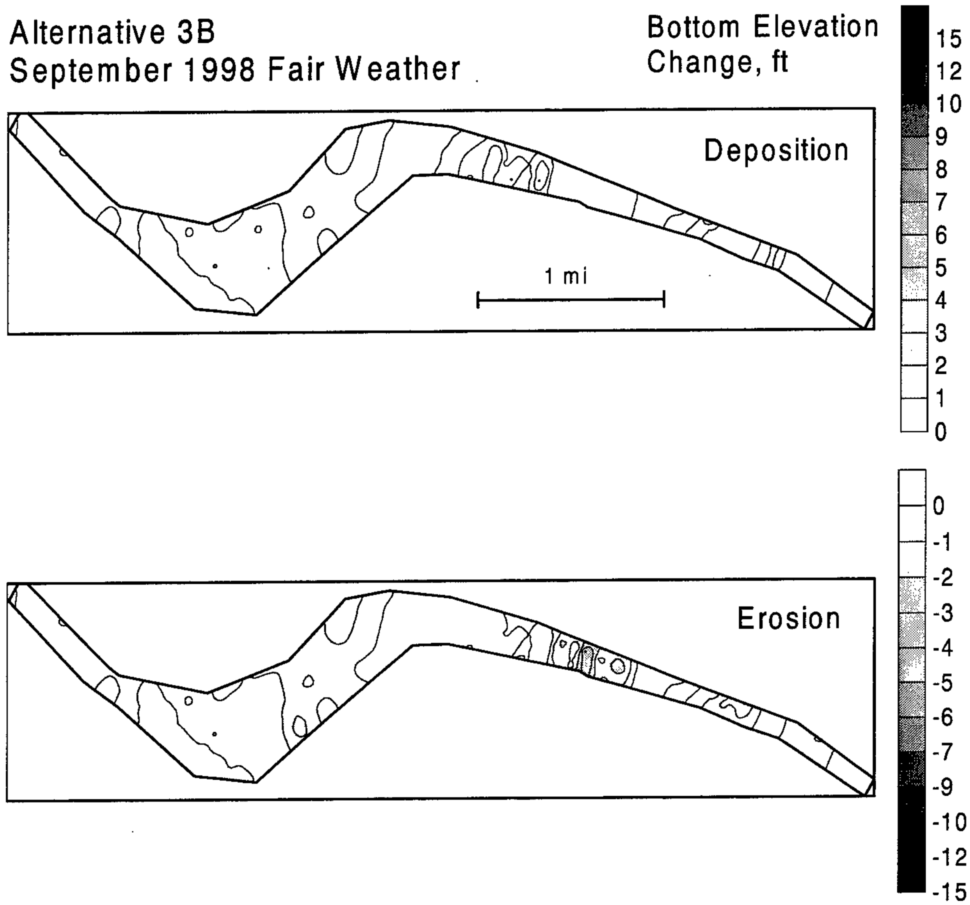

Figure G-38. Calculated deposition and erosion in Alternative 3B channel, September 1998 fair weather

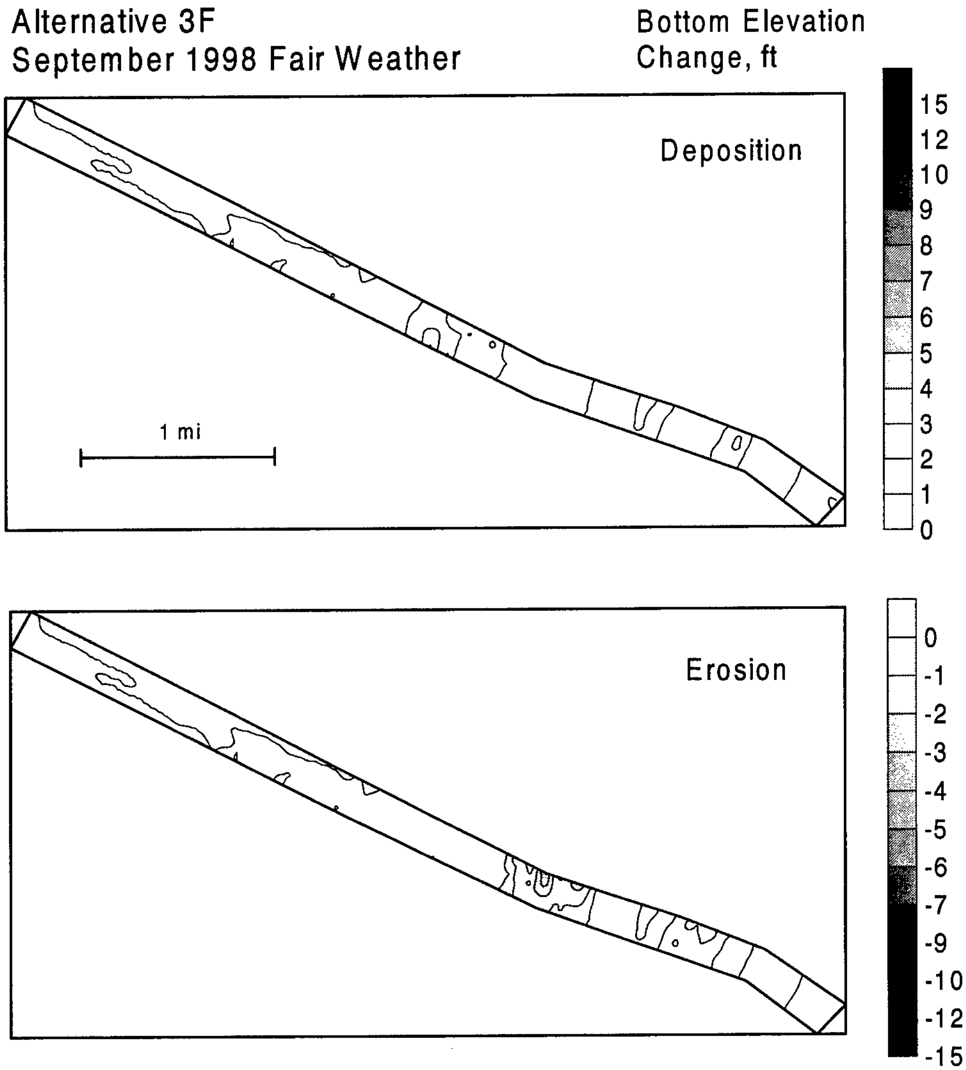

Figure G-39. Calculated deposition and erosion in Alternative 3F channel, September 1998 fair weather

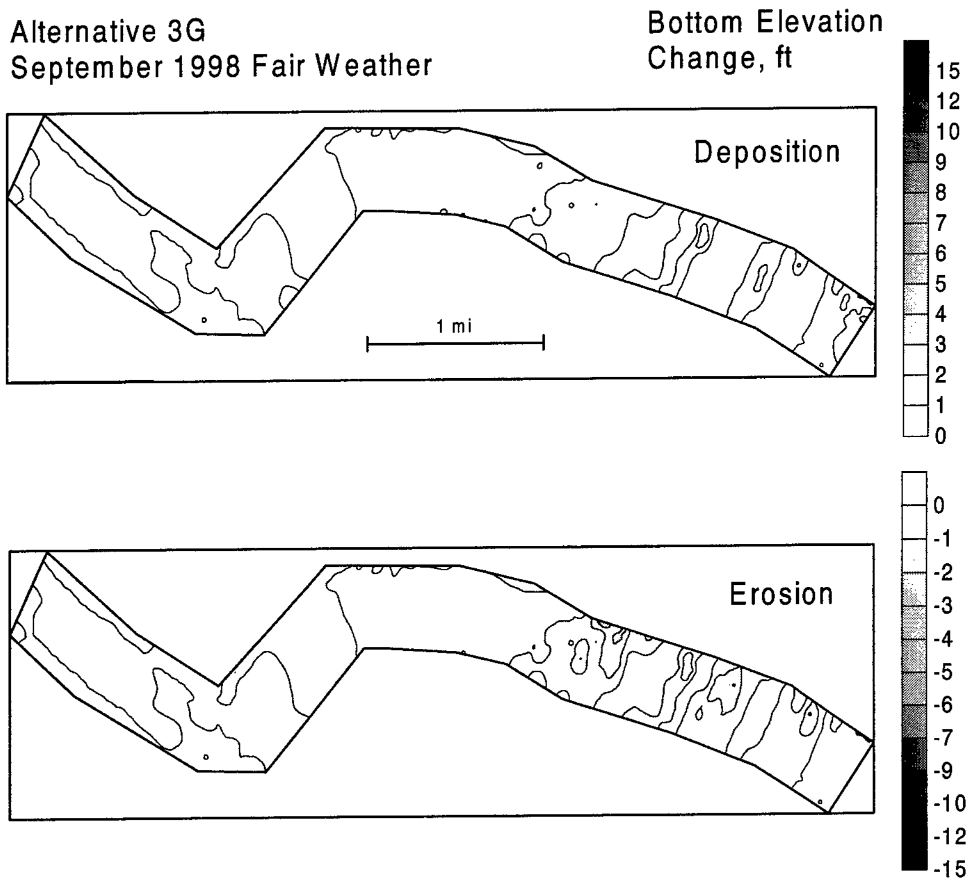

Figure G-40. Calculated deposition and erosion in Alternative 3G channel, September 1998 fair weather

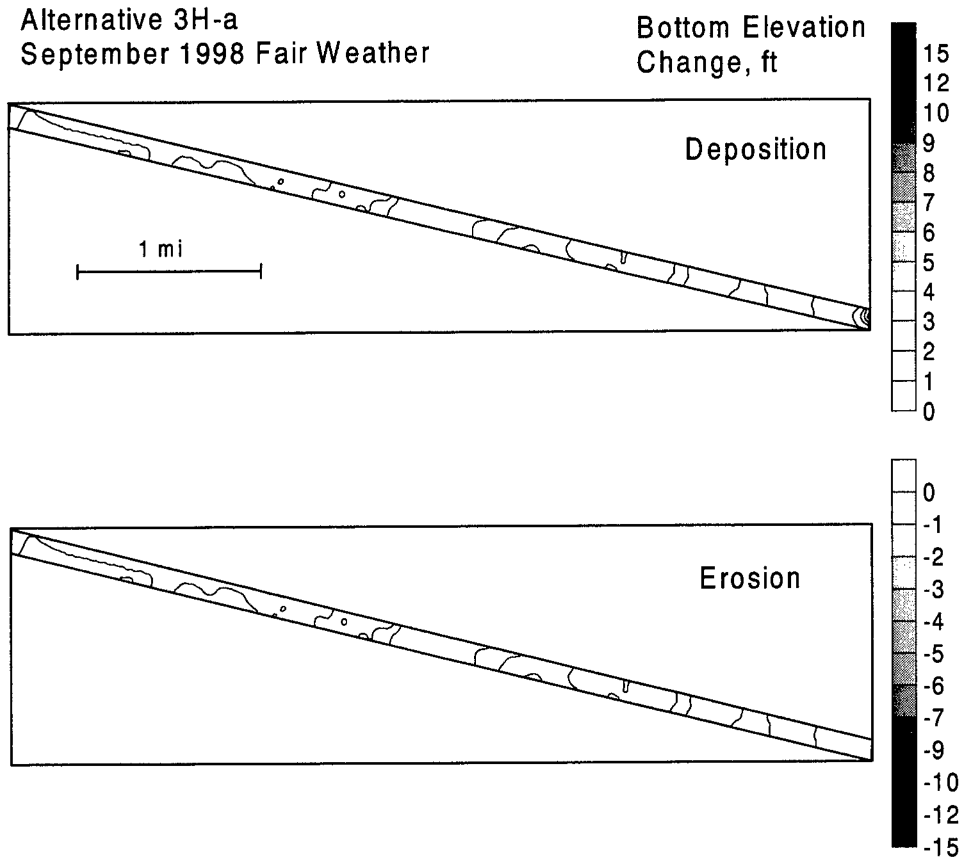

Figure G-41. Calculated deposition and erosion in Alternative 3H-a channel, September 1998 fair weather

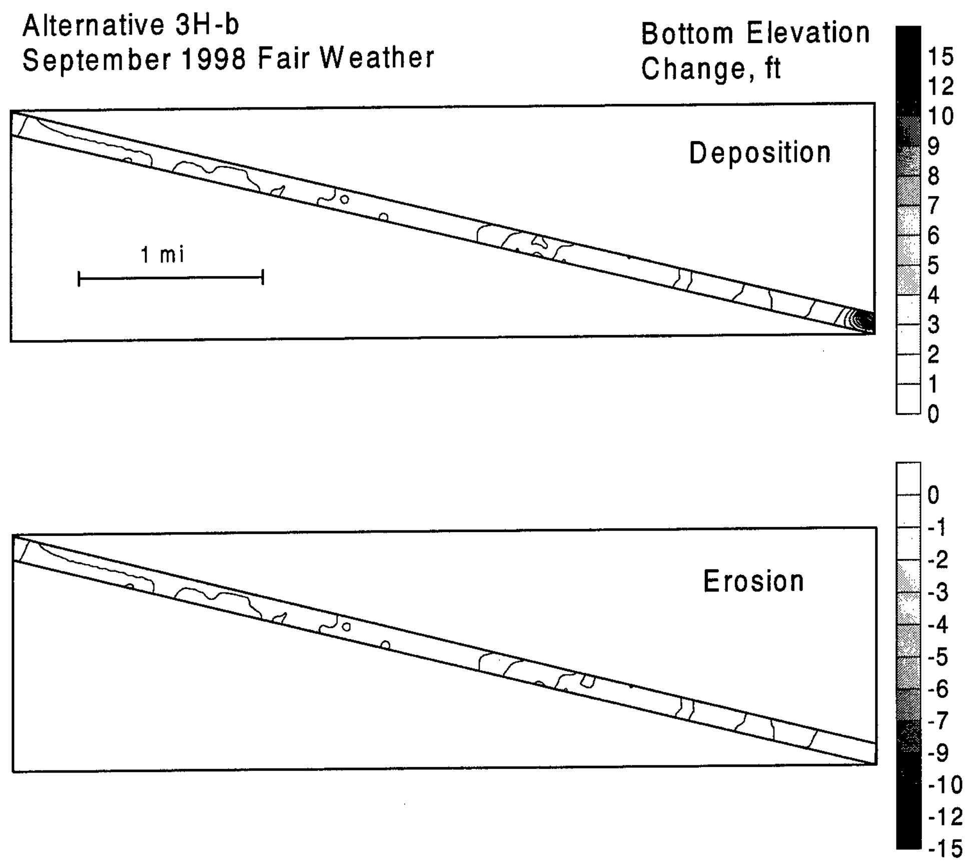

Figure G-42. Calculated deposition and erosion in Alternative 3H-b channel, September 1998 fair weather

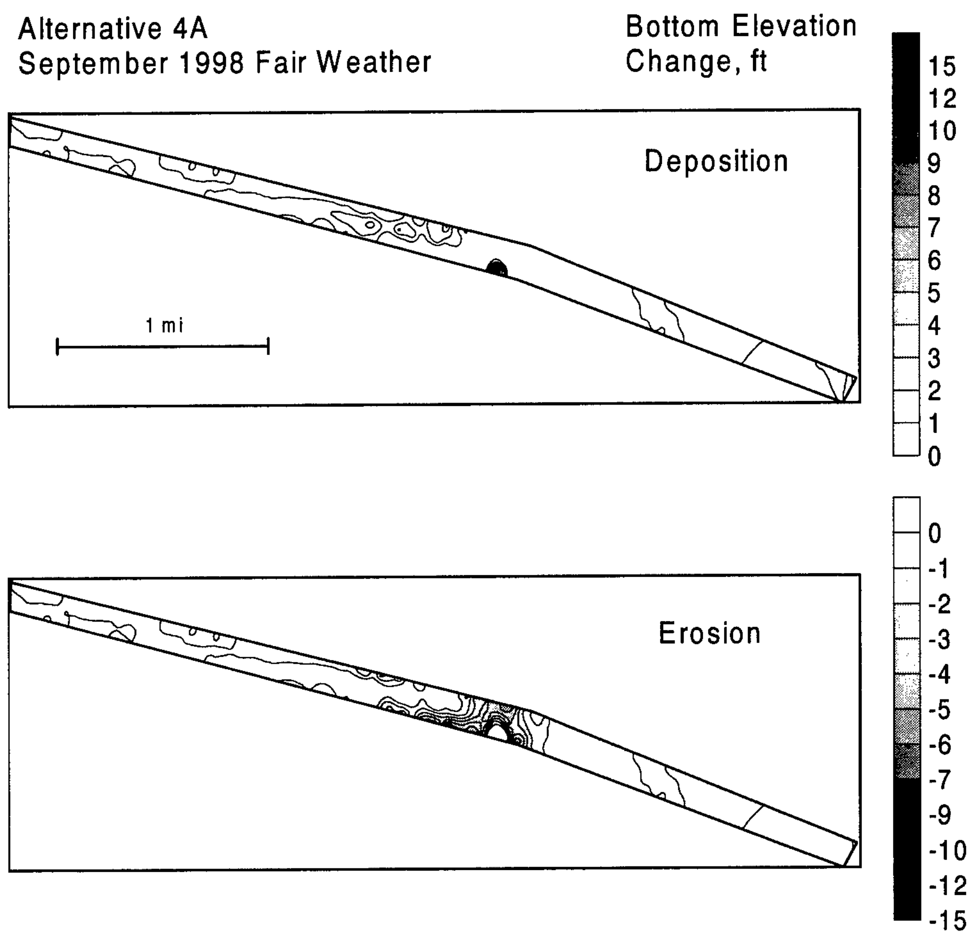

Figure G-43. Calculated deposition and erosion in Alternative 4A channel, September 1998 fair weather

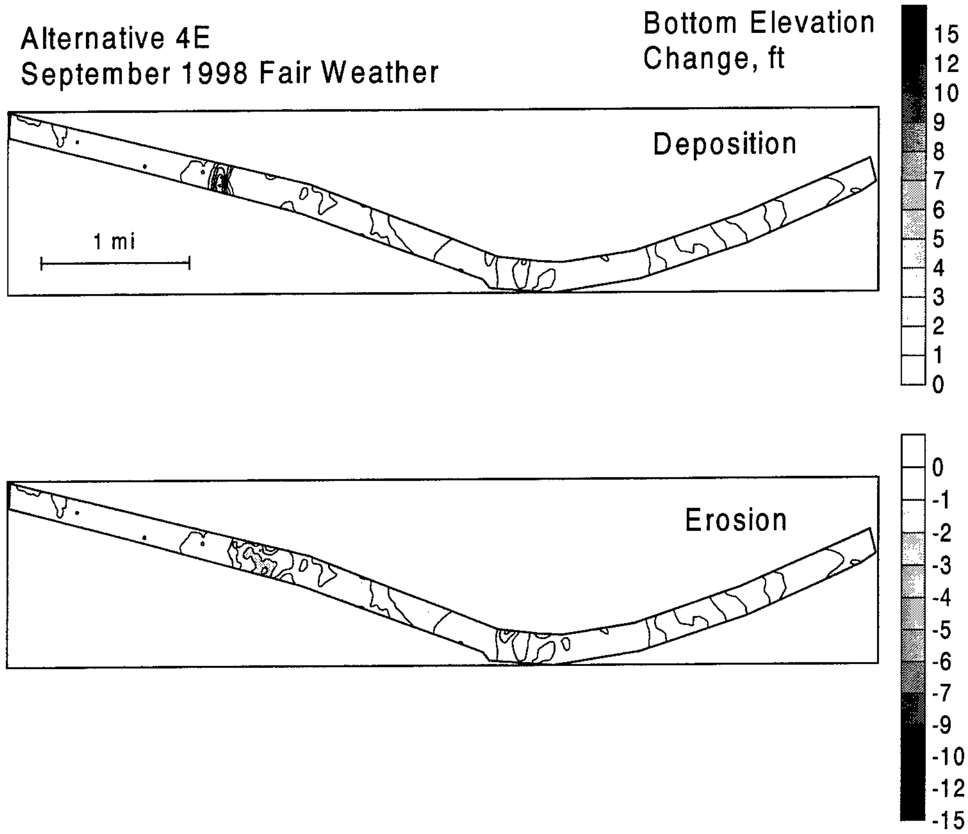

Figure G-44. Calculated deposition and erosion in Alternative 4E channel, September 1998 fair weather

Appendix H
Willapa Bay Dredged Material Disposal Sites Evaluation[1]

Introduction

This appendix identifies and evaluates potential dredged material disposal sites for each of the screened Bar Navigation Channel dredging alternatives at the entrance of Willapa Bay. The dredging alternatives, as identified in Chapter 2, are located in areas primarily consisting of clean sand. Currently designated sites for disposal of dredged material, as well as previously permitted disposal site locations, are identified. Potential new rehandling disposal sites for beneficial use of the dredged material are investigated. The cost of disposal, possible environmental impacts, site capacity, and disposal equipment requirements are also estimated for each of the disposal site alternatives. This appendix develops information for use in applying disposal site criteria in the selection process only, and is not intended as a substitute for a thorough evaluation in appropriate environmental documentation.

Willapa Bay Dredged Material Historical Disposal Sites

Historically, maintenance dredging in the Willapa Bay area was conducted for deep-draft commercial navigation (Outer Bar and Willapa River navigation channels) and for shallow-draft commercial fishing and pleasure vessels (Bay Center channel, Tokeland Marina and channel, Nahcotta Marina). Clean, sandy shoal materials are encountered on the Willapa Bar and at the bay center entrance channel. A chronology of known Federal dredging activities is listed in Table H-1 and displayed in Figure H-1.

Dredging for deep-draft commercial navigation first occurred in 1928. Deep-draft channel-maintenance dredging of the Outer Bar and Willapa River channels was conducted regularly for log ship navigation until 1974 on the bar and until 1976 on the Willapa River. Bar channel dredging was performed by Government

[1] Written by Dr. Vladimir Shepsis and Mr. R. Shane Phillips, Pacific International Engineering PLLC, and Mr. Hiram T. Arden, U.S. Army Engineer District, Seattle, Seattle, WA.

Table H-1
Chronology of Federal Dredging, Willapa Harbor, Washington

Date	Willapa River, cu yd	Bay Center, cu yd	Bay Center Entrance, cu yd	Bar Channel, cu yd	Inner Harbor, cu yd	Toke Point, cu yd	Dredge Type	Comments: Dredging, Disposal, Location, ***Dredge Name***
Oct20-Jun21	335,800						Pipeline	291,240 cu yd at South Fork and 44,560 cu yd at North Fork, *Oregon*
Jul21-Jun22	371,510						Pipeline	371,510 cu yd at North Fork, *Oregon*
Jul24-Jun25	192,850						Pipeline	158,450 cu yd at South Fork and 3,400 cu yd at North Fork, *Oregon*
Feb-Apr 28	216,100						Pipeline	*Oregon*
Jul -Oct 31	369,436						Pipeline	Inner Harbor & River, *Oregon*
Jul31-Jun32				1,242,329			Hopper	*Mackenzie* with 1,450 cu yd bin capacity
Jul-Sep 32				661,766			Hopper	*Mackenzie*
Aug-Oct 33				822,301			Hopper	*Mackenzie*
Jan-May 34	552,826						Pipeline	
Jul-Sep 34				722,319			Hopper	*Mackenzie, Michie* each with 1,450 cu yd bin capacity, built in 1924
Jul-Sep 35				843,655			Hopper	*Mackenzie Michie*
Jul 35-Feb 36	1,062,973						Pipeline	Upland NE of Mailboat slough, new work at Narrows Cutoff
Jul-Oct 36				2,175,502			Hopper	*Mackenzie, Michie*
Oct 36					130,778		Hopper	Lower River, *Kingman* with 1,450 cu yd bin capacity, built in 1924
Jul-Sep 37				1,056,815			Hopper	*Mackenzie, Michie*
Jan-Feb 38					169,306		Hopper	River channels, *Pacific* with 500 cu yd bin capacity
Jul-Sep 38				489,029	454,233		Hopper	8,025 cu yd at Toke Pt shoal and 446,208 cu yd River *Michie, Pacific*
May 40				49,315	141,761		Hopper	141,761 cu yd at Toke Pt shoal, *Kingman*
Sep-Oct 40				77,423			Hopper	*Michie*
Sep 42[1]					71,614		Pipeline	*Multnomah* 24-in. diam
Ave 1930-1940				***814,045***				
Apr -Jul 46	572,050						Pipeline	Upland, *Robert Gray*
Jul 49-Jun 50				1,298,634	498,892		Hopper	Includes 6,500 cu yd at Toke Pt. shoal *Michie, Pacific*
Apr-Aug 49		56,465					Pipeline	Upland, new work at Bay Center channel
Jun 51[1]				342,196	349,990		Hopper	Ocean & open water within bay, *Michie, Pacific*

(Sheet 1 of 5)

Table H-1 (Continued)

Date	Willapa River, cu yd	Bay Center, cu yd	Bay Center Entrance, cu yd	Bar Channel, cu yd	Inner Harbor, cu yd	Toke Point, cu yd	Dredge Type	Comments: Dredging, Disposal, Location, *Dredge Name*
Jul 51		3,903		308,136			Hopper	*Mackenzie*
May-Jul 51		12,403					Clamshell	Open water, deep water off Goose Point
Apr 53					52,226		Hopper	Willapa River at Mailboat Slough, *Pacific* with 500 cu yd bin capacity, built in 1937
Sep-Oct 53				30,277	164,792		Hopper	Ocean & open water within bay, *Davison* with 720 cu yd bin capacity, built in 1945
May-Jun 55					128,302		Hopper	Willapa River, *RossellL* with 1,450 cy bin capacity, built in 1924
Mar-Jun 55		27,958					Clamshell	Open water, deep water off Goose Point
Nov-Dec 55					134,408		Hopper	Willapa River, *Pacific*
May-Jun 56				186,882	72,182		Hopper	Ocean, *Rossell*
May-Jun 57				21,570			Hopper	Ocean, *Rossell*
May-Jun 57					220,912		Hopper	Open water, disposal within estuary, *Pacific, Rossell*
Apr-May 57					89,990		Hopper	Open water, disposal within estuary
Aug 57				234,613			Hopper	Ocean, *Davison*
Nov-Dec 57					85,316		Hopper	*Pacific*
Nov 57-Jan 58					126,147		Hopper	Open water, disposal within estuary
Feb 58		13,882	400				Clamshell	Open water
Aug-Sep 58				253,000			Hopper	Ocean
Sep 58					100,746		Hopper	Open water, disposal within estuary
Dec 58-Mar 59	588,816					184,405	Pipeline	Upland, new work construction of Toke Point & 437,235 cu yd at Nahcotta, *Mccurdy* with 26-in. diam
Sep 59-May 60				138,367	106,823		Hopper	Ocean & Open water, disposal within estuary, *PACIFIC*
Sep 59				98,780			Hopper	Ocean
Apr-May 60				86,700			Hopper	Ocean
Jul-Nov 60					99,901		Hopper	Open water, disposal within estuary, *Pacific, Davison*
Jul-Aug 60				221,827			Hopper	Ocean, *Davison*
Sep-Nov 60	307,957						Pipeline	Upland
Jun-Sep 61		23,889					Clamshell	Open water, disposal off Goose Point
Jul-Aug 61				282,036			Hopper	Ocean, *Davison*

(Sheet 2 of 5)

Table H-1 (Continued)

Date	Willapa River, cu yd	Bay Center, cu yd	Bay Center Entrance, cu yd	Bar Channel, cu yd	Inner Harbor, cu yd	Toke Point, cu yd	Dredge	Comments: Dredging, Disposal, Location, *Dredge Name*
Apr 62				31,037	28,000		Hopper	Open water, disposal within estuary, *Pacific*
Apr 62				28,215			Hopper	Ocean
Jul 62-Jun 63				393,913			Hopper	Ocean, *Pacific, Davison*
Nov 62-Jan 63	509,161						Pipeline	Upland
Feb-Jun 63					133,500		Hopper	Open water, deep water off Johnson Slough, *Pacific*
Jun-Jul 63				102,375			Hopper	Ocean
Aug 63				179,366			Hopper	Ocean
Feb-Mar 64					116,990		Hopper	Open water, disposal within estuary, *Pacific*
Jan-Mar 64		22,050					Pipeline	Upland, *Mole* with 12-in. diam
Jan-Mar 64						11,742	Pipeline	Upland, *Mole* with 12-in. diam
Aug 63-Jun 64				278,212			Hopper	Ocean, *Davison*
Jan-Jun 65	695,311						Pipeline	Upland and open water, *Malemute* with 16-in. diam
Jan-Jun 65			39,652				Hopper	Open water, deep water off Goose Point
Apr 65				23,650			Hopper	Ocean
Apr-May 65				692,653	99,044		Hopper	Open water, disposal within estuary, *Davison*
Jul-Aug 65				238,450			Hopper	Ocean, *Davison*
Jul-Aug 65					14,920		Hopper	Open water, disposal within estuary, *Davison*
Mar 66					29,725		Hopper	Open water, disposal within estuary, *Pacific*
Jul-Aug 66					125,200		Hopper	Open water, disposal within estuary
Jul-Aug 66				303,000			Hopper	Ocean, *Davison*
Jan-Feb 67					66,950		Hopper	Open water, disposal within estuary
1967[2]		29,970	23,002				Pipeline	Open water disposal north of Goose Pt.
May-Jun 67	424,752						Pipeline	Upland
Aug 67					12,410		Hopper	Open water, disposal within estuary
Jul-Aug 67				340,600			Hopper	Ocean, *Harding* with 2,680 cu yd bin capacity
Aug-Sep 67		28,000					Pipeline	Bay Center Upland
		12,000						Open water, Palix River
Feb 68					15,900		Hopper	Open water, disposal within estuary

(Sheet 3 of 5)

Table H-1 (Continued)

Date	Willapa River, cu yd	Bay Center, cu yd	Bay Center Entrance, cu yd	Bar Channel, cu yd	Inner Harbor, cu yd	Toke Point, cu yd	Dredge Type	Comments: Dredging, Disposal, Location, *Dredge Name*
Feb 68				48,700			Hopper	Ocean
Jul-Sep 68				410,000			Hopper	Ocean
Jul-Aug 68					170,650		Hopper	Open water, disposal within estuary
Jul-Sep 69					80,070		Hopper	Open water, disposal within estuary
Jul-Sep 69				610,000			Hopper	Ocean
Oct 69-Jan 70	648,641						Pipeline	Upland
Jan-Feb 70					95,260		Hopper	Open water, disposal within estuary
Jun 70					11,480		Hopper	Open water, disposal within estuary
Jul-Sep 70		28,906					Pipeline	Upland, *Hoquiam* with 12-in. diam
Jul-Sep 70						16,490	Pipeline	Upland
Jul-Sep 70					189,500		Hopper	Open water, disposal within estuary
Jul-Sep 70				324,100			Hopper	Ocean
Nov 70-Jan 71					88,790		Hopper	Open water, *Pacific*
Feb 71			4,000				Hopper	Open water, off Goose Point, *Pacific*
Jul-Oct 71					122,107		Hopper	Open water, disposal within estuary
Jul-Oct 71				340,000			Hopper	Ocean
Jun 72				80,000			Hopper	Ocean, *Pacific*
Jun 72			4,600		28,000		Hopper	Open water, disposal within estuary
Jun-Oct 72	850,237						Pipeline	Upland
Jul-Aug 72				193,500			Hopper	Ocean
Jul-Aug 72					286,000		Hopper	Open water, disposal within estuary
Apr-Jun 73				163,000			Hopper	Ocean, *Pacific*
Apr 73				26,000			Hopper	Ocean, *Pacific*
Apr 73					28,400		Hopper	Open water, disposal within estuary, *Pacific*
			2,600				Hopper	Open water, *Pacific*
Feb-Mar 74					60,710		Hopper	Open water, *Pacific*
			18,800					
Jun 74				42,000			Hopper	Ocean, **last bar O&M for deep draft**, *Pacific*

(Sheet 4 of 5)

Table H-1 (Concluded)

Date	Willapa River, cu yd	Bay Center, cu yd	Bay Center Entrance, cu yd	Bar Channel, cu yd	Inner Harbor, cu yd	Toke Point, cu yd	Dredge Type	Comments: Dredging, Disposal, Location, ***Dredge Name***
Ave 1957-1974				***220,146***				
Jun 74						14,052	Pipeline	Upland, *Hoquiam* with 12-in. diam
Jul-Aug 74		29,434					Pipeline	Upland, *Hoquiam* with 12-in. diam
Aug-Nov 76	268,197						Pipeline	Upland, **last river channel O&M for deep draft,** *Husky* with 20-in. diam
May-Jul 77		20,348	27,989				Clamshell	Open water, Goose Point site
1978[3]		13,300					Hopper	Open water north of Goose Pt., *Pacific*
Aug-Sep 84			3,063				Clamshell	Open water, Goose Point site
		60,382						Open water, Goose Point site
Jun-Jul 87						27,749	Clamshell	Open water, Cape Shoalwater
Jun-Jul 89		33,159					Clamshell	Open water, Goose Point site
Jul-Aug 91						27,135	Clamshell	Open water, Cape Shoalwater
Jul-Aug 93			28,714				Clamshell	Open water, Goose Point site
		22,152						Open water, Goose Point site
Oct-Dec 95			38,327				Clamshell	Open water, Goose Point site
1-9 Aug 97[3]				80,000			Hopper	Open water, **agitation** by sidecast test dredging, *Yaquina* with 1,125 cu yd bin capacity, built in 1981
Sep-Oct 98						27,647	Clamshell	Open water, Goose Point site
			8,000				Clamshell	Open water, bucket sidecast

[1]Start date unknown.

[2]Start and end dates unknown.

[3]Agitation dredging assignment cut short by hull damage to *Yaquina* incurred during dredging on bar.

NOTE: Bar channel dredging assignments often included contingency areas such as the inner harbor which could be dredged when it was too rough on the bar. For the purpose of this tabulation, the hopper dredge volumes reported for the inner harbor include: Toke Point shoals, the North River shoal, and Willapa River. Volumes reported for the Willapa River generally represent hydraulic pipeline dredging work. Disposal method for hopper and clamshell equipment assumes bottom dump discharge unless specified otherwise. Of the early hopper dredges, only the *Pacific* was equipped for the option of sidecasting dredged material which was not found effective for Willapa bar channel sands.

(Sheet 5 of 5)

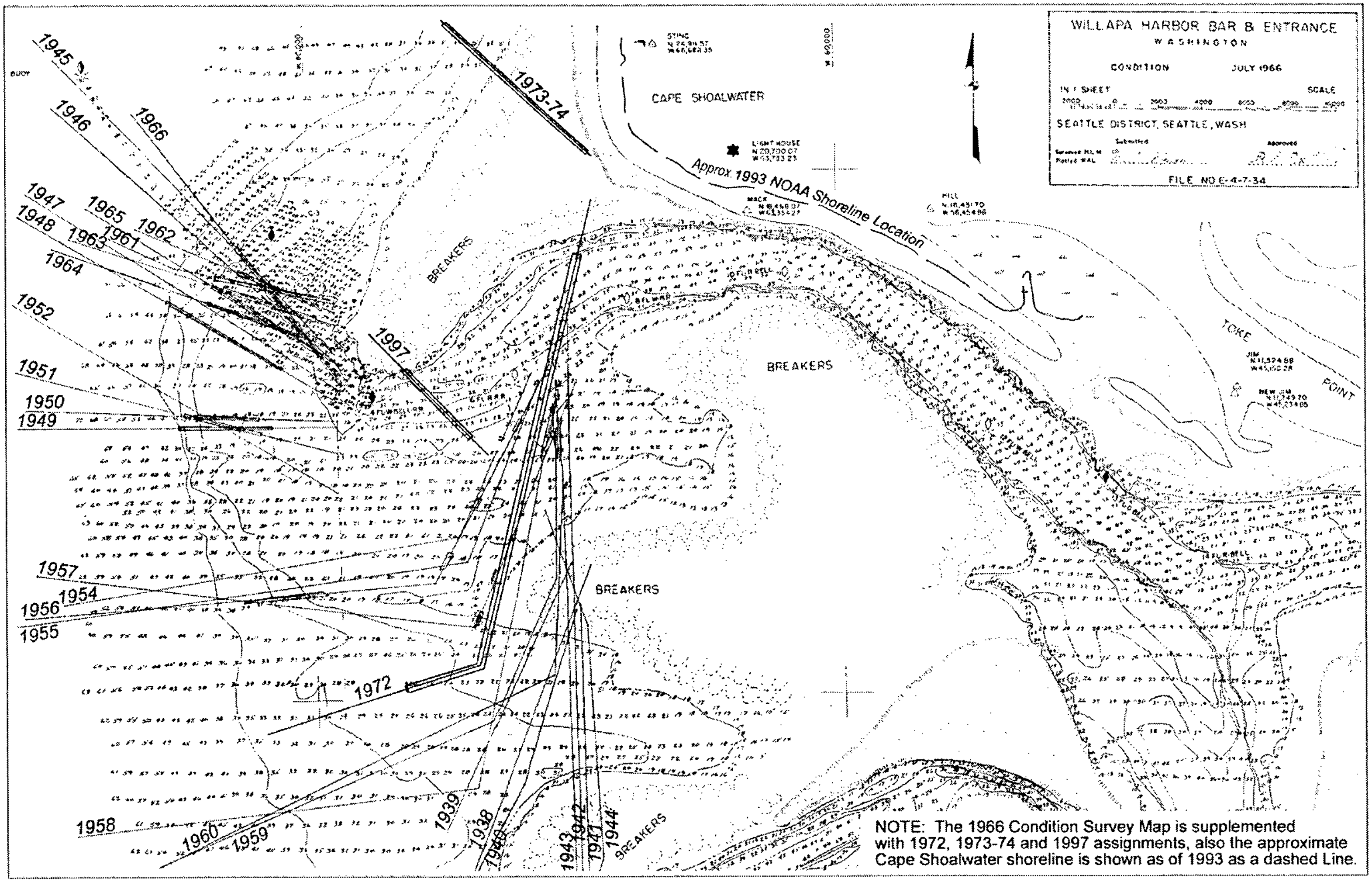

Figure H-1. Historical bar channel dredging assignments (approximate locations and alignments). Note: This 1966 condition survey map is supplemented with 1972, 1973, 1994, and 1997 assignments

hopper dredge equipment with bottom dump disposal at open-water sites within Willapa Bay or at the ocean. The early hopper dredges were small and underpowered compared to present hopper dredge equipment. Hopper agitation dredging was used in the mud sediments of the Willapa River, but agitation dredging was not feasible for heavier bar channel sands.[1] Willapa River dredging was performed using large pipeline dredge equipment with disposal at port-furnished disposal sites. The volume of annual maintenance dredging on the Outer Bar channel varied significantly. Estimated averages of annual maintenance dredging for two periods between 1930-1940 and 1957-1974 are 815,000 cu yd and 220,000 cu yd, respectively.

Dredging for shallow-draft commercial fishing vessels has continued to the present time at Bay Center, Tokeland, and Nahcotta. Dredging volumes for the Tokeland and Bay Center sites have been approximately 25,000 to 60,000 cu yd every 3 to 4 years. The Nahcotta Marina required maintenance dredging only once since initial construction of the project in 1958. Maintenance dredging of the shallow draft projects is restricted to small-business dredge equipment because only the smaller equipment can access these projects with reasonable tide delays relative to channel dimensions.

Disposal of dredged material historically had occurred at various open-water disposal sites both inside and outside the Willapa Bay area and at upland sites. Five disposal sites have been identified (not including disposal sites along the Willapa River) that were designated for the open-water and upland disposal of dredged material from various navigation projects in Willapa Bay. In addition to disposal associated with the navigation project, the Washington State Department of Transportation (WDOT) dredged and disposed 350,000 cu yd of sand along the North Cove shoreline (beach nourishment) in 1998. This disposal was part of a shoreline protection measure for State Route 105 (SR-105). Of the above-mentioned five disposal sites, only the North Cove shoreline is presently authorized for the disposal of dredged material within Willapa Bay. This site is located near "Washaway Beach" along the North Cove shoreline. Both historical and presently authorized Willapa Bay disposal sites are described below.

Cape Shoalwater open water disposal site

Location. The Cape Shoalwater disposal site was located within the North Channel of Willapa Bay, approximately 1 n.m. east of the Willapa Bay entrance. Historically, this disposal site location was referenced to the U.S. Coast Guard (USCG) Entrance Buoy No. 10, which was discontinued. The designated site was defined by the Washington Department of Natural Resources (WDNR) as an 1,800-ft diameter disposal zone with a center located in the north channel thalweg, immediately northeast of USCG Buoy No. 10 (Environmental Protection Agency 1995). The buoy has served as a navigation aid for the North Channel along the North Cove (Washaway Beach). Because of channel migration and changes in available depth near the buoy, the USCG relocated Buoy 10 constantly. As a result, the physical location of the disposal site has moved, consistent with the buoy movement (Figure H-2). Construction of the

[1] Mr. Ted C. Hunt, U.S. Army Engineer District, Portland, Master, hopper dredge *Yaquina*, October 1999.

SR-105 Emergency Stabilization Project caused the USCG to discontinue their Buoy No. 10 and required the installation of a yellow "D" obstruction marker buoy to be installed near the underwater rock structure.

Authorization. This disposal site is presently not authorized for disposal of dredged sediments. Permits authorizing the disposal of dredged sediments at this Buoy No. 10 location expired on 7 July 1999. Prior to the expiration of the Buoy No. 13 site in 1978, disposal of sediments within this disposal site was allowed only during flood tide, (Figure H-2). Since 1978, tidal stage restrictions have been removed as a condition for use of the Cape Shoalwater site. The WDNR is finalizing permit applications to Pacific County and the State of Washington for the continued availability of an open-water site in the vicinity of Cape Shoalwater. The proposed Cape Shoalwater open-water site location is northeast of USCG Buoy No. 13. The location of Buoy No. 13 changes and was last charted by the USCG in 45 ft of water at mean low lower water (mllw) in September 1999. All references to depth and elevation in Appendix H are referenced to the plane of mllw, and the annotation mllw will be omitted.

Source of dredged material. This site has been used for disposal of dredged material primarily from the Tokeland Federal entrance channel and the Port's Marina maintenance dredging.

Goose Point open-water disposal site

Location. The Goose Point Disposal Site is located 3.6 n.m. east of Leadbetter Point within the Nahcotta channel of Willapa Bay (Figure H-2). The coordinates of the site are latitude 46°38′51″N and longitude 123°59′54″W. This circular-shaped disposal site has a 1,800-ft diameter.

Authorization. Prior to 7 July 1999, this site was authorized for disposal of suitable dredged sediments from Bay Center, Nahcotta, and Toke Point Marina. Permits for the disposal of dredged material have expired and are presently under reapplication for authorization.

Source of dredged material. This site has been used for disposal of dredged material primarily from the Bay Center channel; however, in 1998 dredged material from the Toke Point Marina was allowed to be disposed at the Goose Point open-water site to avoid potential conflicts with the WDOT underwater rock dike construction project at Cape Shoalwater. Maintenance dredging of the Nahcotta Marina has also considered the Goose Point site as an alternative, but rather, utilized an upland site owned by the Port of Peninsula.

Interim ocean disposal site

Location. This site was located approximately 3.5 n.m. west of the entrance to Willapa Bay. The dumping line site was approximately 5 n.m. in length along a line between latitude 46°44′N, longitude 124°10′W and latitude 46°39′N, longitude 124°09′W. The location of the dumping line was based on the approximate locations of historic hopper dredge ocean disposal relative to the natural migrations of the bar channel.

Authorization. The U.S. Environmental Protection Agency (EPA) designated the site as an interim open-water disposal site in 1977.[1] This site is not authorized for disposal of dredged sediments. The interim site did not receive final designation or approval from the EPA.

Source of dredged material. This site had been available for disposal of dredged material from Outer Bar Channel dredging if the need were to arise for safe shallow-draft navigation. No hopper maintenance dredging was performed after 1974 or after the proposed interim site designation in 1977.

Buoy No. 13 disposal site

Location. This site was located where the Willapa River enters the bay southwest of Toke Point (Figure H-2). Historically, the location of this disposal site was referenced to the USCG Entrance Buoy No.13. As described for the Cape Shoalwater disposal site, this site was subject to the regular relocation of the navigation buoy resulting from the North Channel migration.

Authorization. Prior to being discontinued in 1978, use of this site was restricted to ebb tide only and is not presently authorized as a disposal site. An area northeast of USCG Buoy No. 13 is proposed to be the new location for a Cape Shoalwater open-water disposal site because the USCG Buoy No. 10 has been discontinued. Relocation of the Cape Shoalwater site is necessary for barge and hopper dredge equipment safety to avoid the vicinity of the WDOT submerged rock structure. Water depths in the area of Buoy No. 13 were measured at 45 ft by the USCG. Estimated depths in the proposed disposal site are between 15 and 50 ft. The shallower water is based on an estimated minimum feasible depth that dredged material disposal equipment can access in conjunction with beach nourishment and beneficial uses of dredged material.

Source of dredged material. This site was used from 1968 to 1978 as an ebb tide alternative disposal area to the Cape Shoalwater Buoy No. 10 site, which was restricted to flood tide use. This site served primarily for the disposal of material dredged from the Tokeland Marina. Sandy materials to be dredged in the maintenance of the Bay Center entrance channel are being considered for beneficial uses in conjunction with beach nourishment.

SR-105 Emergency Stabilization Project (ESP) beach nourishment site

Location. The site is located at the shoreline starting at the east side of the SR-105 groin structure and extending westward about 3,200 ft and is approximately 3.5 n.m. east of the Willapa Bay entrance (Figure H-2).

Authorization. This site was authorized for placement of dredged material under WDOT SR-105 ESP that was constructed during 1998. The existing permits authorizing placement of dredged material along the SR-105 ESP shoreline will expire in March 2001.

[1] Personal communication with Mr. John Malek, EPA Region 10 office, describing status of interim open ocean disposal site, September 1999.

Source of dredged material. A permitted 350,000 cu yd dredged material borrow site is located within the North Channel 0.5 n.m. east of the underwater rock dike.

Comments. Dredging and placement of dredged material were restricted to hydraulic dredging with direct disposal only above elevation 3.0 ft. Approximately 350,000 cu yd of sandy material was placed along the shoreline between SR-105 milepost 20.4 and 21.0 as a temporary feature of the construction project.

Environmental Issues

Disposal site location and method of disposal are central factors for evaluating the economic feasibility of potential disposal sites. Disposal sites were evaluated relative to environmental permitting and coordination requirements and known resource agency requirements, and environmentally sensitive areas and biological resources.

Environmental permitting and coordination requirements

The overlapping authorities of Federal, State, and local jurisdiction entities regulate environmental permitting of dredging and disposal of dredged materials. The permitting process for any of the dredged material disposal sites listed in this study will require the interagency coordination among the following agencies:

a. U.S. Army Corps of Engineers (Corps).

b. U.S. Environmental Protection Agency (EPA).

c. Washington Department of Ecology (WDOE).

d. Washington Department of Natural Resources (WDNR).

e. Washington Department of Fish & Wildlife (WDFW).

f. Local Jurisdictions such as Pacific County and the Shoalwater Bay Tribe.

The EPA and other Federal and State agencies become involved through the Corps' permitting process. The permitting authorities for dredging and dredged material are described in the following paragraphs.

U.S. Army Corps of Engineers. Under Section 404 of the Clean Water Act (CWA), the Corps is required to regulate the disposal of dredged or fill material in the waters of the United States. The purpose of the CWA is to restore and maintain the chemical, physical, and biological integrity of the waters of the United States. The CWA prohibits the discharge of dredged or fill material except in compliance with Section 404.

Under Section 10 of the Rivers and Harbors Act of 1899, the Corps is required to regulate the dredging, disposal, and performance of most work in the navigable waters of the United States.

Corps Projects. Corps projects, which involve discharge into navigable waters, are subject to the same evaluation procedures as non-Corps projects: the Corps issues a Public Notice and may schedule a public hearing(s). Generally, a

Corps project has a local sponsor (Port of Willapa Harbor in the present situation) who, by Congressional authorization, is required to furnish dredged material disposal areas. Sediment sampling is performed in proposed dredging areas to characterize dredged materials to determine suitability for open-water disposal. The local sponsor frequently performs ancillary maintenance dredging work that is combined into the Corps' Public Notice. If a WDNR open-water disposal is used (Cape Shoalwater or Goose Point), the local sponsor obtains the appropriate site use permission from WDNR.

Environmental Protection Agency. The EPA has several roles under Section 404 of the CWA. First, EPA has the responsibility for developing the Section 404(b)(1) Guidelines in conjunction with the Corps. Second, EPA reviews the Corps' Public Notice and gives comments to the Corps. Third, the EPA Administrator, via Section 404(c), may prohibit the specification of a discharge site, or restrict its use if it is determined that discharge would have an unacceptable adverse effect on fish and shellfish areas, municipal water supplies, wildlife, or recreational areas. In addition, under 40 CFR 230.80, EPA and the Corps may jointly provide advance identification of suitable or unsuitable disposal sites.

Section 102 of the Marine Protection, Research, and Sanctuaries Act requires EPA, in consultation with the Corps, to develop environmental criteria for the evaluation of proposed ocean disposal activities.

Washington Department of Ecology. The WDOE has the responsibility for certifying compliance with Section 401 of the CWA. This certification is required from any applicant for a Federal permit to conduct an activity that may result in discharge into State waters.

The WDOE's Public Notice for the CWA and the Coastal Zone Management Act (CZMA) certification are mailed together with the Corps' Public Notice. If a project involves disposal in water, data on the sediments are required. After the review of the available data, the application for Water Quality Certification and CZMA consistency determination is approved or denied. The project applicant is responsible for providing a copy of the Water Quality Certification and CZMA decision to the WDNR and the Corps before these agencies, respectively, can issue a disposal site use permit and Section 10 and 404 permits.

The WDOE coordinates the final overall State of Washington response to the Corps' Public Notice. To fulfill this responsibility, WDOE sends a "State Response Letter" to the Corps after receiving comments from interested State agencies. This letter describes the State's concern, and recommends approval or denial of the Corps permit.

All dredging projects also require a Water Quality Modification from WDOE. This grants the applicant a short-term modification from compliance with the State of Washington's water quality standards. The modification will be issued simultaneously with the Water Quality Certification for a minimum 1-year period. Longer periods are common.

The WDOE also establishes guidelines for State and local administration of the Washington Shoreline Management Act (SMA). The WDOE ensures that permits issued by local Governments are consistent with the SMA. If a permit

does not appear to be consistent, WDOE may appeal the permit to the Shorelines Hearing Board.

Washington Department of Natural Resources. The WDNR acts as the proprietor and manager of State-owned aquatic lands, which include all subtidal lands within 3 miles of the Washington coastline. For each open-water disposal site located on State lands, WDNR is required by the SMA to obtain a Shoreline Substantial Development (SSD) permit from the local jurisdictions (Pacific County). This permitting process allows for public review at the local level, including private interests and Government agencies. As the SSD permit applicant, WDNR is also the lead agency under State Environmental Policy Act (SEPA).

The WDNR issues dredged material disposal site use authorizations for each disposal operation. The WDNR sends the proponent an application upon their request. The proponent submits the disposal site use application to WDNR. The WDNR does not issue the authorization until the applicant has obtained all required regulatory permits (i.e., the Corps permit, WDOE Water Quality Certification, and WDFW Hydraulic Project Approval).

Corps navigation projects are exempt from WDNR's dredged-material disposal site use authorization requirements. For Corps projects having local sponsors, the sponsors must obtain the disposal permit for their berthing area dredging volumes, but not for Federal channel dredging volumes.

Washington Department of Fish and Wildlife. The WDFW has the responsibility to issue a Hydraulic Project Approval (HPA) under a process outlined in Chapter 75.08.012 (The Fisheries Code), 75.20.100 Revised Code of Washington (RCW), and Chapter 220-100 Washington Administrative Code. The WDFW issues most HPAs in saltwater areas. The purpose of the HPA is to protect fish life.

Under RCW 65.20.100, WDFW must approve or deny the HPA application within 45 days of receiving a complete application, providing the project has achieved SEPA compliance. The WDFW accepts the Corps' Public Notice as the application for the HPA.

Local Governments. Under the SMA, local Governments have the responsibility for general land use planning for shoreline development. Local Governments control shoreline land use planning through the issuance of Shoreline Substantial Development permits.

Environmentally sensitive areas

The entire Willapa Bay estuary contains a wide variety of valuable and sensitive aquatic habitat areas. These areas include sand shoals, eelgrass beds, mudflats, oyster beds, and salt marshes that provide habitat for a range of species. More than a half dozen species listed as endangered or threatened under the Endangered Species Act reside or visit for feeding, nesting, or resting at the surrounding beaches and dunes. Environmentally sensitive areas of the Willapa Bay estuary were determined for the SR-105 ESP Environmental Assessment (Federal Highway Administration and WDOT 1997) and are shown in Figure H-3.

During construction of the SR-105 ESP beach nourishment, resource agencies required a minimum one-half mile buffer between the edge of Sand Island and any proposed construction activities. Similar restrictions may apply to other islands in Willapa Bay.

Potential sensitive areas are the former Cape Shoalwater location of the Willapa National Wildlife Refuge (NWR) and Leadbetter Point Park. NWR had two locations along Washaway Beach prior to being lost to erosion. The first was located halfway between SR-105 ESP rock dike and the beach nourishment borrow site. The second site was located at the outer bar of the North Channel just offshore from Cape Shoalwater. The SR-105 ESP features were required to be outside of the NWR boundaries. Similar restrictions may apply to disposal actions associated with channel dredging.

Biological Resources. Biological and environmental assessments were conducted for the SR-105 ESP. These documents were reviewed and the following biological resources for the Willapa Bar area were identified:

a. Brown pelican: Currently on Endangered Species List. Critical habitat located at Sand Island. No construction activities were allowed within one-half mile of the island.

b. Dungeness crab: Crab present within Willapa Bay at the open-water disposal site locations. Mitigation of impacts caused by dredging and disposal were required.

c. Snowy plover: Currently on the Endangered Species List. Nesting habitat located along Leadbetter Point and Gunpowder Island.

d. Marine fish: Juvenile salmon migration period within Willapa Bay from 1 March to 14 June. Adult salmon migration period during 16 August to 15 October.

e. Herring: Some locations within Willapa Bay may require shutdown of construction activities from 1 February to 14 March for the herring spawning period.

Resource agency requirements

The existing environmental documentation for the SR-105 Emergency Stabilization Project, Willapa Harbor 1997 Test Hopper Agitation Dredging Project, and the Tokeland Marina Maintenance Dredging were reviewed for applicability to this project. These are possible restrictions on the disposal of dredged sediment, which could apply to all of the evaluated disposal sites. Possible restrictions for each of the disposal site alternatives are as follows (EPA 1995):

a. Beach nourishment: No disposal of dredged material above the 30-ft depth from 1 March to 15 June of any year to avoid the adult salmon migration period and possible mitigation.

b. North Channel disposal: Possible restrictions on disposal of dredged material from 16 August to 15 October of any year (adult salmon migration period).

c. Sediment nourishment site: No disposal of dredged material above the 30-ft depth from 1 March to 15 June of any year to avoid the juvenile salmon mitigation period.

d. Agitation dredging: Unknown.

Dredging alternatives may be limited by restrictions on Hopper dredging from 16 August to 15 October of any year.

Development of Disposal Site Alternatives

A range of disposal site alternatives was developed to accommodate placement of dredged material from each of the bar navigation channel dredging design alternatives. The development of the disposal site alternatives was evaluated based upon the following criteria: technical feasibility of disposal, cost of disposal, environmental impact, permit acquisition process, site capacity, and equipment requirements.

Assumptions

Alternatives for disposal sites were developed under the following assumptions:

a. The Port of Willapa Harbor is the local sponsor for all features of the Federal navigation project except for the Nahcotta boat basin, which is sponsored by the Port of Peninsula. As a local sponsor, the ports are required to furnish all lands, easements and rights-of-way needed for the dredged material disposal.

b. Dredging will be conducted during summer to early fall. Dredging and disposal must be performed during 1 year.

c. The sediment dredged from the entire bar navigation channel dredging alternatives is suitable for open-water disposal as well as for beneficial use. The Willapa Bay area is classified as a low ranking area for presence of chemicals of concern (U.S. Army Engineer District, Seattle 1994). Materials dredged from Willapa Bar, according to the Unified Soil Classification System (USGS), ranges from poorly graded beach sands with less than 5 percent passing the No. 200 sieve (SP), and sands with more than 12 percent passing the No. 200 sieve (SM) to fine-grained and well-sorted sand on the outer bar. Very poorly graded clayey silts are present in Willapa River, and very poorly graded silty clays in the marinas (Jackson, Allen, and Lowell 1997).

d. Willapa Bay hydrodynamic conditions at the disposal site locations are based on the data from the SR-105 ESP Monitoring Program from 1997 through 1999, and the data collected for the Willapa Bay Navigation Project Study from fall 1998 to summer 1999.

e. The part of the North Channel in the vicinity of the SR-105 project has changed its alignment since the project was constructed. The channel was relocated toward the south end of the dike, and a scour hole formed at the toe of the dike. Seaward from the SR-105 project, a channel has

developed indicateing a potential new westerly branch formation (Figure H-2). This branch is an active branch of the channel. However, further development of the branch depends on the dynamics of the existing, northerly channel branch. Closure (filling) of the northerly branch of the channel may contribute to the rate of development of the westerly channel branch.

f. Both direct beach and nearshore placement of dredged material can help reduce the rate of erosion along the North Cove shoreline. The volume of beach nourishment should not exceed 350,000 cu yd/year, as authorized for the previous project. The nearshore berm should be placed at a depth shallower than 20 ft to provide wave attenuation and simultaneously minimize sand loss to the deep-water channel. Crest elevation of nearshore berm should be above a depth of 5 ft, but not higher than 0.0 ft so that all placed material is located on the slope in the vicinity of a wave-breaking zone. A high level of energy dissipation characterizes this slope. Dredged material placed at this area will provide a benefit by reducing wave energy that drives longshore and cross-shore sediment transport.

g. Formation of tidal flats at the entrance of Willapa Bay (similar to that at Sand Island, locally called Deadman's Island) occurs periodically following a breach in the North Cove bar. During this geomorphologic cycle, a significant amount of sand is released from the North Cove bar and migrates into Willapa Bay under transport by tidal currents and waves. Disposal of a large amount of dredged material at certain areas of the entrance of the bay (feeder zone) may simulate post-breach conditions and result in formation of tidal flats. Location of the placement zone would be selected based on the analysis of numerical hydrodynamic modeling and available depth for hopper dredge access. The disposal site is considered to be dispersive. Waves and currents will disperse dredged material placed at the site. The rate of dispersion depends on disposal site location and technology of dredged material placement. The preliminary and final designs will be required to address specific dispersivity issues for each of the disposal sites.

h. Because of variability of the South Channel alignment and depth limitation, disposal at the South Channel has not been considered.

i. Hopper dredging with or without cutterhead dredging is the preferred method of dredging for each of the design alternatives. Final combination of hopper and cutterhead dredging is dependent on site conditions of dredging and disposal site. For development of disposal site alternatives, hopper dredging was categorized into three classes: large class (6,000+ cu yd) hopper dredges with a loaded draft of 27 ft, medium class (3,300 to 3,800 cu yd) with a draft of 20 ft, and small class (1,500 to 2,000 cu yd) with a draft of 15 ft.

j. All distances referenced from the Outer Bar were measured from the center of dredging alternatives to each of the disposal sites.

Based on these assumptions, a list of disposal site alternatives was developed and preliminarily screened relative to feasibility, cost, beneficial use, and disposal technology. Preliminary screening reduced the number of disposal site

alternatives to 12 sites that are described in the following paragraphs. These sites are identified in Figure H-4 and summarized in Table H-2.

Beach nourishment at SR-105

Location. This disposal site (referenced in Table H-2 as Alternative A) is located at the existing SR-105 ESP beach nourishment site and approximately 3.5 n.m. inland from the outer bar of the North Channel. Placement of dredged materials would occur between elevations of +3 and +10 ft. Disposal at this location would require the use of a hydraulic dredge or a direct pump-out (from a hopper dredge) with 1,500-ft of submerged pipeline. A direct pump-out operation would require the use of either a mooring barge or a mooring buoy to be positioned along the north edge of the North Channel for the hopper dredge to connect to the submerged pipeline. Wave and currents at the disposal site are assumed to be within the operating parameters of the equipment from June to September.

Authorization. This site is authorized under the WDOT SR-105 ESP construction permits. The existing permits authorizing placement of dredged material along the SR-105 ESP shoreline will expire in March 2001.

Comment. Dredged material placement at this location would provide beneficial use by reducing the volume of SR-105 beach nourishment maintenance dredging. Placement of dredged material at this site may be possible any time of the year except between 1 March and 15 June. Minimal adverse impacts to biological resources would result from this alternative; therefore, mitigation might be avoided. The recipient of this beneficial use of the dredged sand will have to pay the incremental cost exceeding the cost of open-water disposal.

Beach nourishment along North Cove

Location. This disposal site (referenced in Table H-2 as Alternative B) is located along the shoreline of North Cove, approximately 1 mile west of the SR-105 ESP site and 2.5 n.m. inland from the outer bar of the North Channel. Disposal of dredged material would be similar to that described for SR-105 ESP Beach Nourishment.

Authorization. This site is not authorized for disposal of dredged material under existing permits and has not been used as a disposal site in the past.

Comment. Placement of dredged material would provide beneficial use by slowing the erosion of the shoreline along North Cove. This site would be subject to the same environmental constraints as identified in Alternative A. Additionally, impacts to Dungeness crab may require mitigation for losses associated with burial. The recipient of this beneficial use of the dredged sand will have to pay the incremental cost exceeding the cost of open-water disposal.

Table H-2
Development of Alternatives

Disposal Site Alternatives (Depth of Placement, ft)	Relative Cost	Status of Authorization/ Permitting	Capacity cu yd	Equipment Requirements	Feasibility	Beneficial Use Possible
A – Beach Nourishment (elev. +3 to +10)	High	Currently authorized	360,000	Small Hopper with Pumpout	Medium – increased cost to pump, beneficial use	Yes
B – Beach Nourishment (elev. +3 to +10)	High	Not authorized	780,000	Large Hopper with Pumpout	Medium – increased cost to pump, beneficial use, restricted const. period	Yes
C – Nearshore Berm West (depth 20 and shallower)	Med.	Not authorized,	500,000	Small Hopper Dredge	Low – shallow water/high cost, restricted const. period	Yes
D – Nearshore Berm East (depth 20 and shallower)	Med.	Not authorized	330,000	Small Hopper Dredge Cost	Low – shallow water/high cost, restricted const. period	Yes
D-1 – Beach Nourishment East (elev. +3 to +10 and higher)	High	Not authorized	200,000	Small Hopper Dredge Cost	Medium – increased cost to pump, beneficial use, restricted const. period	Yes
E – North Channel (depth 70)	Low	Authorization expired	1,600,000	Large Hopper with direct disposal	High – low cost, beneficial use	Yes
F – Goose Point (depth 20-30)	Med.	Authorization expired Permit renewal pending	900,000	Medium Hopper with direct disposal	Medium – large haul distance	No
G – Interim Ocean (depth greater than 50)	Low	Not authorized	N/A	Large Hopper with direct disposal	Medium – subject to operation in ocean swell	No
H – Sediment Nourishment (depth 20 to 30)	High	Not authorized	900,000	Small Hopper Dredge	Low – shallow water; wave cond	Yes
I – Middle Channel Agitation	Low	Not authorized		Small Hopper Dredge		No
J – Rehandling	Med.	Not authorized	N/A	Cutterhead Dredge	High to medium	Yes
K – Proposed Cape Shoalwater open water	Low	Pending authorization	N/A	Medium Hopper with direct disposal	High	Yes

Nearshore berm west of SR-105 ESP

Location. The west nearshore berm disposal site (referenced in Table H-2 as Alternative C) is located along the west side of the SR-105 ESP groin and dike structures, approximately 2.7 n.m. inland from the North Channel outer bar. Dredged sediments would be placed initially by hopper dredge or bottom dump barge, followed by pipeline disposal (or other method) above a depth of 20 ft to a maximum depth of 5 ft. Waves and currents at the disposal site are assumed to be within the operating parameters of the equipment from June to September.

Authorization. This site is not presently authorized for disposal of dredged material. The location of this alternative is partially within the historical location of the Cape Shoalwater disposal site referenced to the USCG Buoy No. 10.

Comment. Dredged material placed at this location may provide the benefit of erosion protection for a currently retreating stretch of shoreline. The recipient of the beneficial use of the dredged sand will have to pay the incremental cost exceeding the cost of open-water disposal.

Nearshore berm east of SR-105 ESP groin/dike

Location. The east nearshore berm disposal site (referenced in Table H-2 as Alternative D) is located along the east side of the SR-105 ESP groin and dike structures, approximately 4.3 n.m. inland from the North Channel outer bar. Dredged sediments would be placed initially by hopper dredge or bottom dump barge, followed by pipeline rehandling (or other method) at a depth of 20 ft up to a limiting depth of 5 ft. Waves and currents at the disposal site are assumed to be within the operating parameters of the equipment from June to September.

Authorization. This disposal site is not authorized for disposal. It may provide beneficial use as described in the alternative nearshore berm west of SR-105 ESP. If a beneficial use is identified, the recipient of the dredged sand will have to pay the incremental cost exceeding the cost of open-water disposal.

Beach nourishment east of SR-105 ESP groin/dike

Location. This disposal site (referenced in Table H-2 as Alternative D-1) is located along the east side of the SR-105 ESP groin and dike structures, approximately 4.3 n.m. inland from the North Channel outer bar. Placement of dredged material would be similar to that described for SR-105 ESP Beach Nourishment.

Authorization. This site is not authorized for disposal of dredged material under existing permits and has not been used as a disposal site in the past.

Comment. Placement of dredged material at this site may help to nourish a shoreline dune that was breached during a 1998 extreme tide.

North Channel open water disposal

Location. The North Channel disposal site (referenced in Table H-2 as Alternative E) is located along the North Channel thalweg at the SR-105 ESP, located approximately 2.5 n.m. inland of the outer bar. Dredged material could

be placed at this site by bottom dumping from the hopper dredge in depths as great as 70 ft. Because of the significant bottom changes that are presently occurring, the specific location of disposal would be evaluated immediately prior to the placement of dredged material. General location of the disposal can be assumed to be downstream and upstream of SR-105 underwater dike, and at the south end of the dike where the bottom is presently deepening. The proximity of this disposal site to the underwater dike structure is a potential navigation safety concern.

Authorization. This disposal site coincides with the historical location of the Cape Shoalwater open-water disposal site near USCG Buoy No. 10. The site permits expired on 7 July 1999.

Comment. The North Channel thalweg at the dike location has shifted approximately 2,000 ft to the south since the completion of SR-105 ESP construction and current velocity reduction in this vicinity has been documented. Current velocities increase with distance from the north side slope southward toward the channel. Therefore, placement of dredged material along the north side slope of the North Channel could result in beneficial use by stabilizing the channel side slope and lower portion of the beach and assisting the growth of upper beach profile. Possible impacts to Dungeness crab may require mitigation.

Goose Point open water disposal

Location. The Goose Point open water disposal site (referenced in Table H-2 as Alternative F) is located approximately 2.3 n.m. east of Leadbetter Point and 8 n.m. from the North Channel outer bar. Placement of dredged materials would occur by bottom dumping from a hopper dredge at existing depths greater than 20 ft. Wave and current conditions at the disposal site are within the operating parameters of the dredging equipment that would be used for this alternative.

Authorization. This site is currently designated as the Goose Point open-water disposal site. Coordinates of the proposed disposal site are latitude 46°39'28" and longitude 123°59'46" located in 36 to 60 ft of water depth of the Nahcotta Channel thalweg. Disposal site depths will be greater resulting in an increased disposal site capacity. Previous disposal site coordinates from a dated NOAA chart resulted in shallower disposal site depths and increased concerns about site capacity.[1]

Comment. This disposal site would provide no beneficial use of dredged material. Biological resources impacts would be minimal.

Interim ocean disposal site

Location. This disposal site alternative (referenced in Table H-2 as Alternative G) is an ocean site located approximately 1 mile outside the Willapa outer bar.

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[1] Personal communication with Mr. Ted Benson, WDNR, describing status of disposal site authorization at Goose Point and Cape Shoalwater sites, 13 September 1999.

Authorization. This site is not authorized for ocean disposal and requires Corps and EPA designation and approval of dredged sediments. Placement of dredged material at this location would not provide a beneficial use.

Sediment nourishment site

Location. This disposal site (referenced in Table H-2 as Alternative H) is located inshore from the outer bar along the middle shoal between the North and Middle Channels of Willapa Bay. The site is approximately 1 n.m. inland from the outer bar of the North Channel. A small hopper dredge would be required for this site for bottom dump disposal of dredged sediments at existing depths of 20 to 30 ft.

Authorization. This site is not currently authorized for disposal of dredged sediments under existing permits and is not within one of the designated open-water disposal sites in Willapa Bay. Placement of dredged material at this location would provide some beneficial use. Material placed within the sediment nourishment site would serve as a feeder for the formation of tidal flats for additional habitat. Site designation would require EPA designation and approval.

Middle channel agitation dredging site

Location. This type of disposal (Alternative I) would occur along the alignment of the Middle Channel dredging alternative. Existing depths at the dredging site vary from 5 to 28 ft. Side casting in this location would constitute resuspension of the bottom sediments for dispersal by currents and waves.

Authorization. This site is not currently authorized for disposal of dredged sediments under any existing permits and is not within one of the designated open-water disposal sites in Willapa Bay. This alternative would not provide any beneficial use of dredged material. Site designation would require EPA designation and approval. Additionally, impacts to Dungeness crab may require mitigation for losses.

Rehandling site

Location. This rehandling site (Alternative J) is located approximately 0.5 n.m. east of the SR-105 project at the south bank of the North Channel.

Authorization. This site has been authorized as a borrow site for the SR-105 project. This site has not been designated as a disposal site.

Comment. This site is considered a potential rehandling site for beach nourishment disposal.

Proposed Cape Shoalwater open water-disposal site

Location. The proposed open-water disposal site (Alternative K) is northeast of the USCG Buoy No. 13. The buoy is located in about 45 ft of water. The disposal area location will vary with movements of the buoy. Disposal area

depths vary from 15 to 50 ft. The shallower depths would be associated with potential beach nourishment disposal uses of dredged material.

Authorization. The WDNR is finalizing a SMA application to Pacific County for state approval and designation of a Cape Shoalwater open-water disposal site.

The preceding disposal site information is summarized in Table H-2.

Disposal Sites/Navigation Channel Alternative Combinations

More than one disposal site can be feasible for a dredging alternative. A preliminary attempt was made to assign the disposal site alternatives to the navigation channel dredging alternatives. The list of navigation channel alternatives presented in Table H-2 served as the basis for assigning disposal site alternatives. The matching of channel dredging and disposal site alternatives was a logistical and iterative process that accounted for cost of disposal, schedule of construction, site capacity and volume of dredging, and dredging and disposal equipment compatibility. These preliminary dredging site and disposal site combinations are presented in Table H-3. The cost for the disposal operation was estimated using CEDEP software (Callan 1998) and is also presented.

In developing the cost estimate for dredged material disposal alternative the following major assumptions were used:

a. Placement of dredged material to beach nourishment and nearshore berm is provided by a hopper dredge only.

b. Capacity of a small hopper dredge is 2,100 cu yd. Capacity of a large hopper dredge is 6,000 cu yd.

The identified disposal site, as well the combination of disposal sites and navigation channel alternatives, are preliminary and should be specified during permitting and design. These dredging site/disposal site combinations may be modified after completing numerical modeling and if new information about channel sedimentation and volume of future maintenance dredging becomes available.

Table H-3
Dredging Site/Disposal Site Combination Alternatives

Alternative	Dredging Volume, cu yd	Description	Disposal Sites	Estimated Dredging and Disposal Cost[1] per cu yd
3A	800,000	Dredge primarily on entrance bar, straight out and fixed in position.	A, B, or D-1 – Beach Nourishment	$8.70
			C or D – Nearshore Berm	$6.60
			E – North Channel Disposal	$4.30
3B	200,000	Modified S-curve (moderate curve).	A, B, or D-1 – Beach Nourishment	$8.70
			C or D – Nearshore Berm	$6.60
			E – North Channel Disposal	$4.30
3H-a, 3H-b	1,600,000	Added after 1999 survey to coincide with the straight path seaward from near SR-105 project that had minimal change since 1998 survey. Two variations include existing SR-105 (3H-a) and SR-105 dike raised to -2 ft (3H-b).	A, B, or D-1 – Beach Nourishment	$8.70
			C or D – Nearshore Berm	$6.60
			E – North Channel Disposal	$4.30
4A	2,300,000	Dredge primarily on entrance bar.	F – Goose Point	$6.80
			I – Agitation Side Casting	$3.80
4E	12,600,000	Same as 4A, except dredge to total depth of 38 ft and with width 1,000 ft.	F – Goose Point	$5.90
			I – Agitation Side Casting	$3.50

[1] Each cost estimate is based on disposal of amount shown in second column, though in some cases, it may be advantageous to dispose of different portions of this amount at more than one site in a given year. Cost Estimate does not include mobilization and demobilization. Mobilization and demobilization costs will vary depending on volume of dredged sediment and dredged material disposal technique from $100,000 (small volume of sediment and small dredge to be used) up to $2,000,000 (large volume of dredge material and large dredge to be used).

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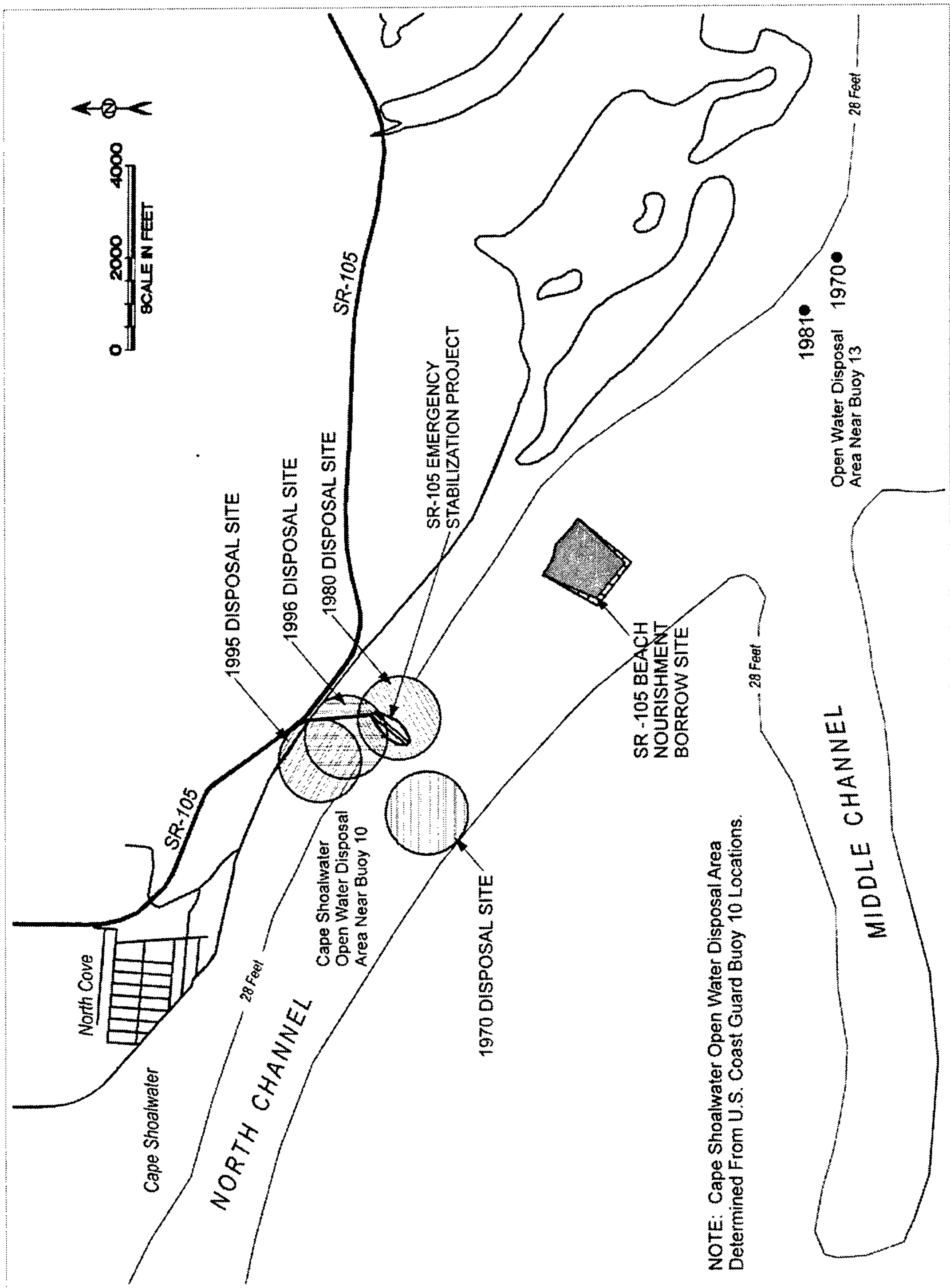

Figure H-2. Willapa Bay navigation project, historical Cape Shoalwater disposal sites near USCG Buoy 10 and Buoy 13

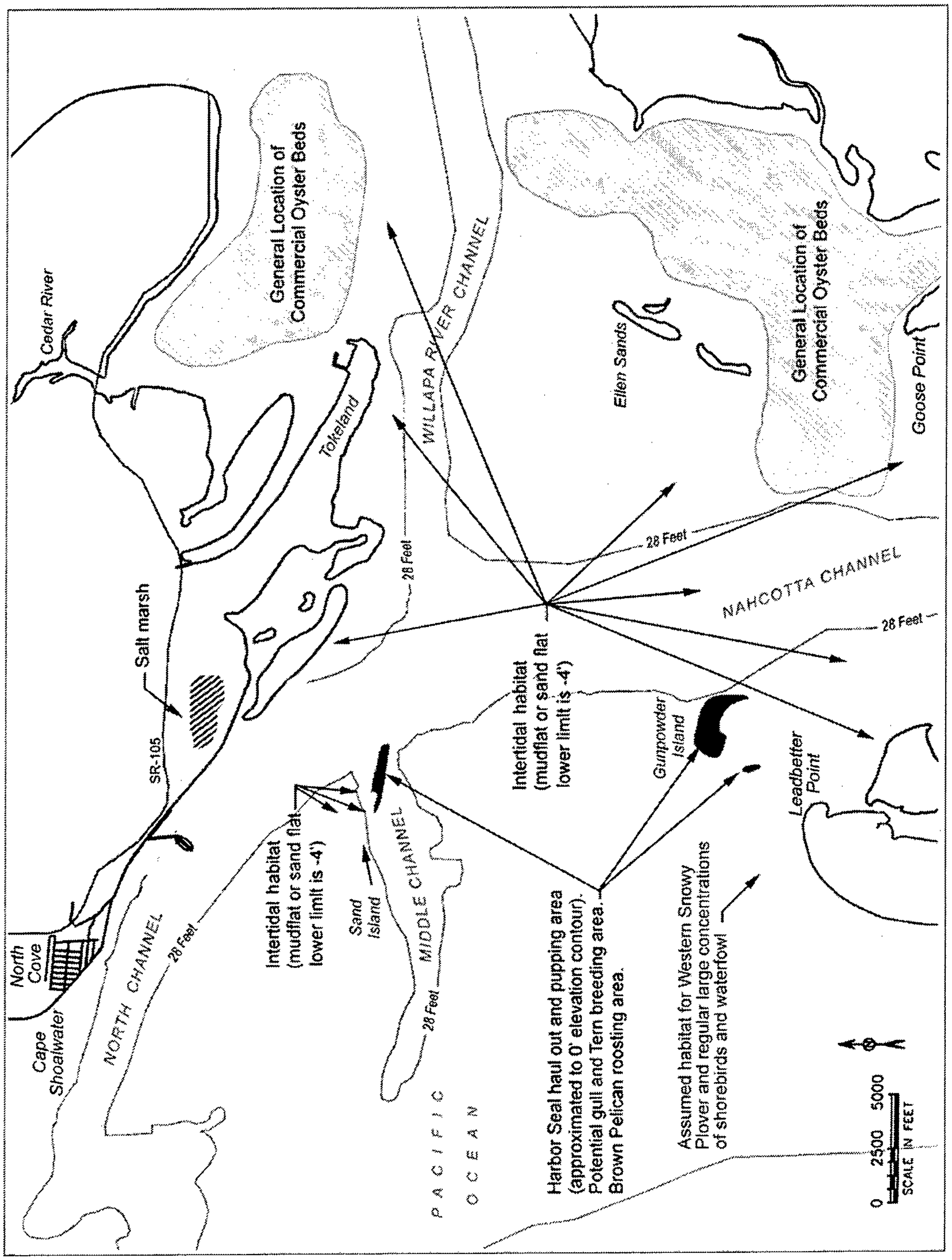

Figure H-3. Willapa Bay navigation project, environmentally sensitive areas

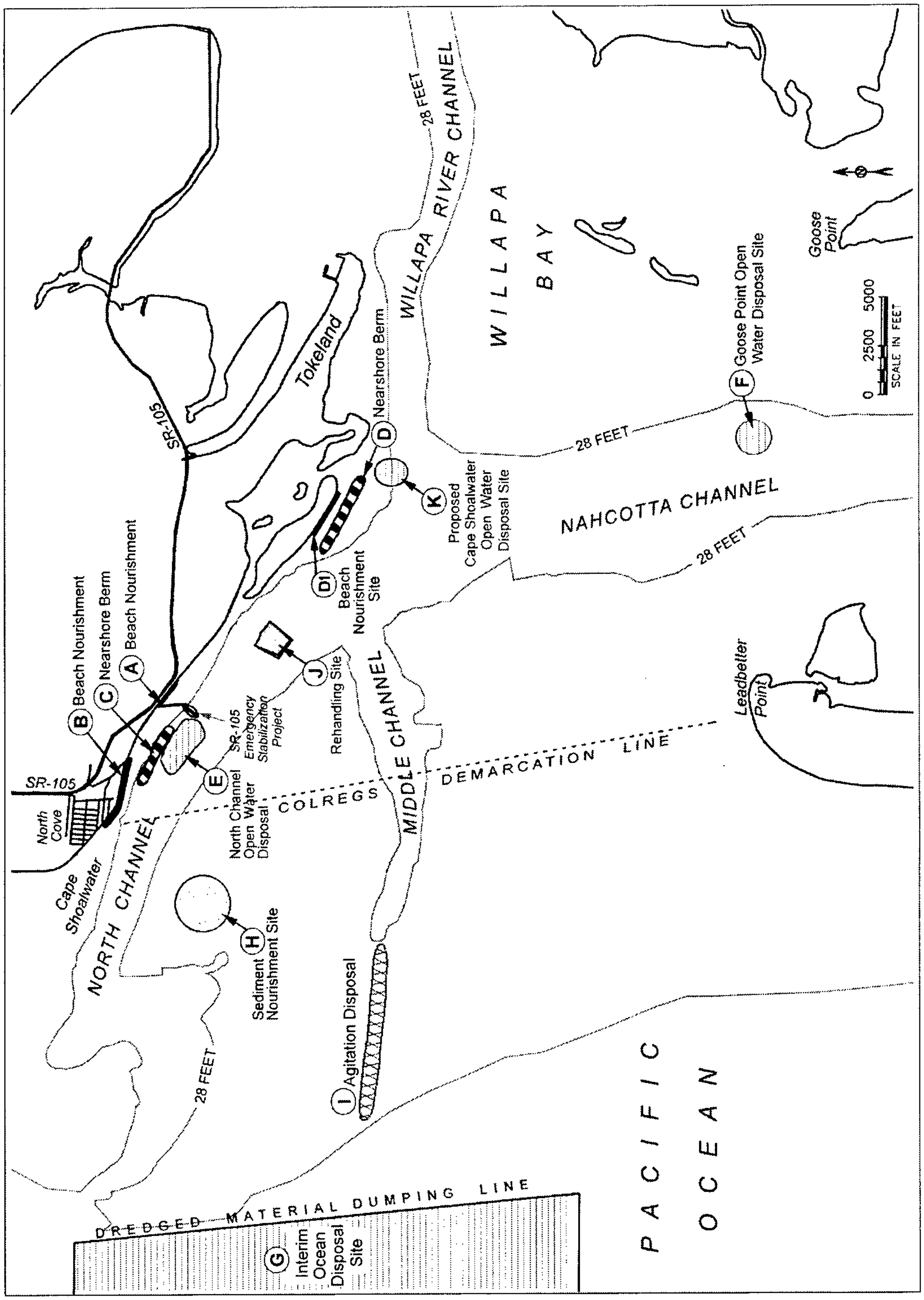

Figure H-4. Willapa Bay navigation project disposal site alternatives

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14. ABSTRACT
The navigation channel reliability monitoring and evaluation study for Willapa Bay, Washington described in this report was performed by the U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory for the U.S. Army Engineer District, Seattle (NWS). The study was established under a Partnering Agreement between the NWS and the Willapa Port Commission for determining the feasibility of maintaining a reliable navigation channel through the Willapa Bay entrance.

Willapa Bay is a large estuarine system located on the southern end of the Washington coast. Its spring or diurnal range tidal prism is one of the largest of all inlets on the coast of the continental United States. The shifting channels at the entrance to Willapa Bay make navigation unreliable, and the local port cannot maintain or attract commercial users. Local interests obtained Congressional support to determine if an economical deep-draft channel can be established through the entrance bar. An economical channel implies a route that can be traversed safely under typical waves and tidal currents.

The study was conducted in a multi-disciplinary approach involving engineering analysis, field measurements, geomorphologic analysis, and numerical modeling of waves, currents, and sediment transport in evaluation of alternative channel designs. These topics are covered in the main text of the report, with additional details and data compilations contained in appendices.

15. SUBJECT TERMS

Cape Shoalwater	Dredging	Geomorphology	Navigation channel	Washington	Willapa Bay
Currents	Field measurement	Navigation	Sediment Transport	Waves	

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